CHAPTER 1 **INTRODUCTION**

Chapter Synopsis: This chapter presents the purposes, objectives, and scope for the 2004 Sewer Master Plan Revision. It also summarizes previous sewer master plans and studies done by the City that are pertinent to the sanitary sewer system.

The 2004 Sewer Master Plan Revision is a re-evaluation of the 2002 Sewer Master Plan (RMC, 2003) and achieves several key objectives, including 1) conduct a wet weather flow monitoring program, 2) conduct a topographical survey of portion of the sewer system, 3) update and calibrate the sewer system computer model under HYDRA Version 6.0, 4) update the potential wet weather conveyance and pumping capacity deficiencies and associated 2002 Capital Improvement Program under existing (as of March 2004), near- (2008) and long- (2018) term conditions using the information from the wet weather flow monitoring and topographic survey data, and 5) provide information required for the City planning and financial efforts.

The City of Milpitas (City), situated 45 miles south of San Francisco, is bordered by Fremont to the north and San Jose to the south. Figure 1-1 shows the location of the City with respect to surrounding communities. It is a part of the Santa Clara County and the "Silicon Valley."

Figure 1-1: City of Milpitas Location[1]

Since its incorporation in 1954, present-day Milpitas has experienced steady growth and development. At the time of incorporation, the City covered an area of 2.9 square miles with a population of 825. Rapid growth began with the Ford Motor Company assembly plant in 1955 and continued with the high technology industry in the 1970's. By 1992, the City had covered a present day area of 13.6 square miles. The United States Census 2000 shows Milpitas having a population of 62,700.

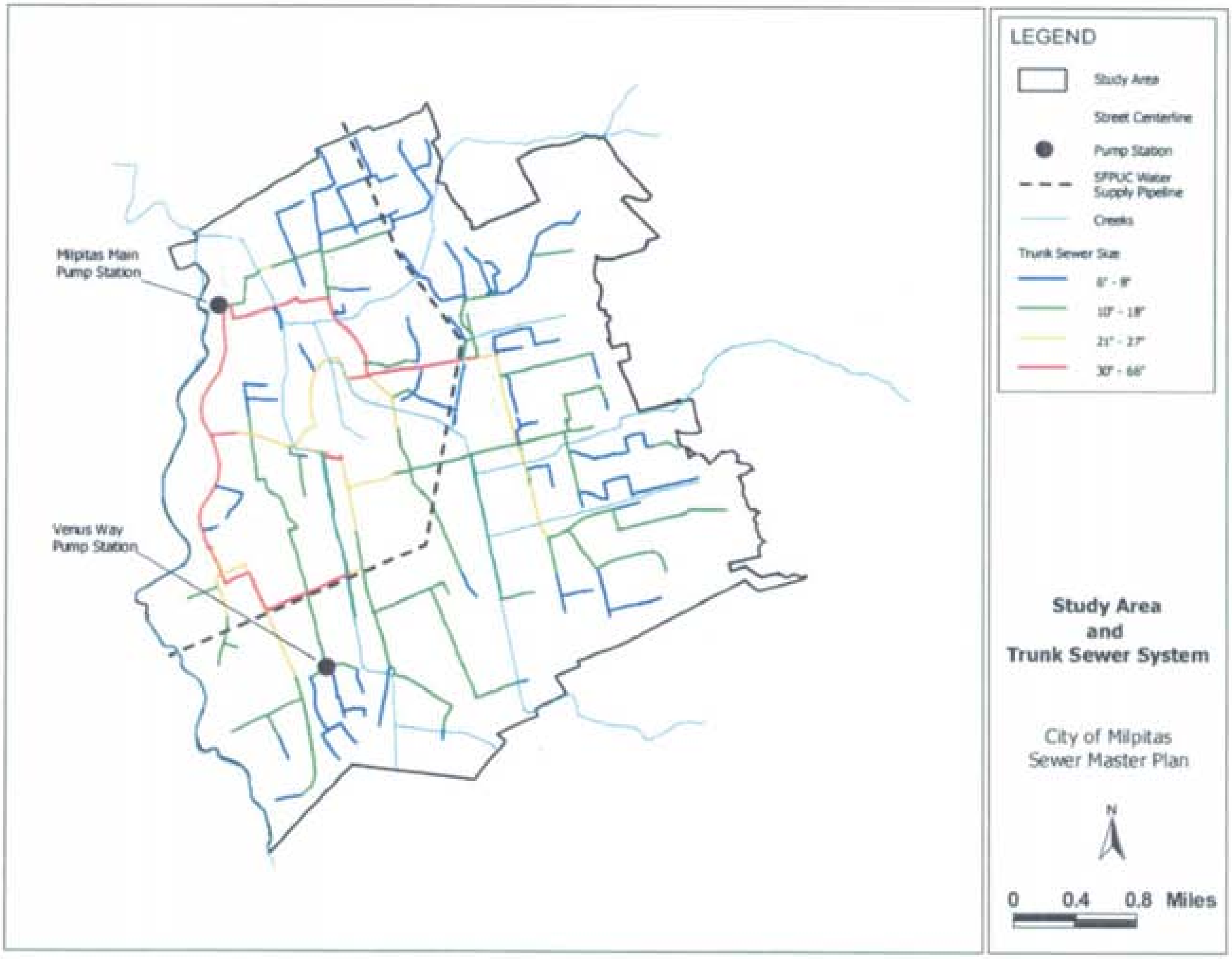
The City has a strong complement of industrial, retail, and housing uses. The City can be divided into two distinct areas consisting of roughly 10.1 square miles on the relatively flat "Valley Floor" to the west and 3.5 square miles on the steep "Hillside" to the east. The Valley Floor areas, extending from Coyote Creek in the west to Piedmont Road, Evans Road and the northerly portion of North Park Victoria Drive in the east, are zoned for industrial, commercial, and residential uses. The Hillside areas are zoned for residential use only. Parks and recreational open spaces are distributed throughout the residential areas.

[1] Map is excerpted from the Expedia.com website at http://www.expedia.com/

The City's urban service area boundary is defined based on the General Plan and General Plan amendments, most importantly the Resolution No. 6796 of the City Council. This 1998 resolution established a new urban service area boundary by prohibiting City services (including sewer service) in areas outside of the urban growth boundary and outside of the City limits. This resolution is applicable until December 2018 (it could be amended under specific circumstances through a General Plan amendment). For the purpose of this Master Plan Revision, it is assumed that the resolution will not be amended before 2018 – the established planning horizon. Figure 1-2 shows the boundaries of the City's urban service area, i.e. this Master Plan study area.

The City's sanitary sewer system collects the wastewater flows from approximately 6,000 acres within the City planning area, serving a population of approximately 63,800 through 860,640 linear feet (LF) of sewers. The City's wastewater flows are conveyed mostly by gravity to the Milpitas Main Pump Station (Main PS), which pumps all the flow to the San Jose/Santa Clara Water Pollution Control Plant (WPCP) through two force mains. A second pump station, located on Venus Way, connects a low-elevation portion of the City to the gravity sewer system. The sewer system includes a number of siphons under the San Francisco Public Utility Commission (SFPUC) water supply pipeline, creeks and highways. Figure 1-2 shows the alignment and size of trunk sewers and the location of the two pump stations. The two force mains between the Main PS and the WPCP are not represented.

Figure 1-2: Master Plan Study Area and City's Trunk Sewer System



1.1 Project Purpose

This Master Plan Revision is a re-evaluation of the 2002 Sewer Master Plan (RMC, 2003), which was an update of the 1994 Sewer Master Plan Update (Corollo Engineers, 1994). The 2004 Master Plan Revision provides information required for the City planning and financial efforts and defines the sanitary sewer system improvements necessary to accommodate the City's future land use development plans to the year 2018, or buildout.

The City of Milpitas Utility Engineering Department is facing several potential planning issues:

1. Not all of the sewer capacity improvements recommended in the 1994 Sewer Master Plan Update have been implemented (see Figure 1-4), and new land use development patterns for the City, such as the Midtown Specific Plan, have since been defined that could stress the system beyond the 1994 assumptions.
2. The City is nearing its wastewater capacity at the WPCP. New development in the City could trigger the need to purchase additional capacity at the plant.
3. The City is experiencing maintenance problems (i.e. need for frequent cleaning) with some of its siphons.

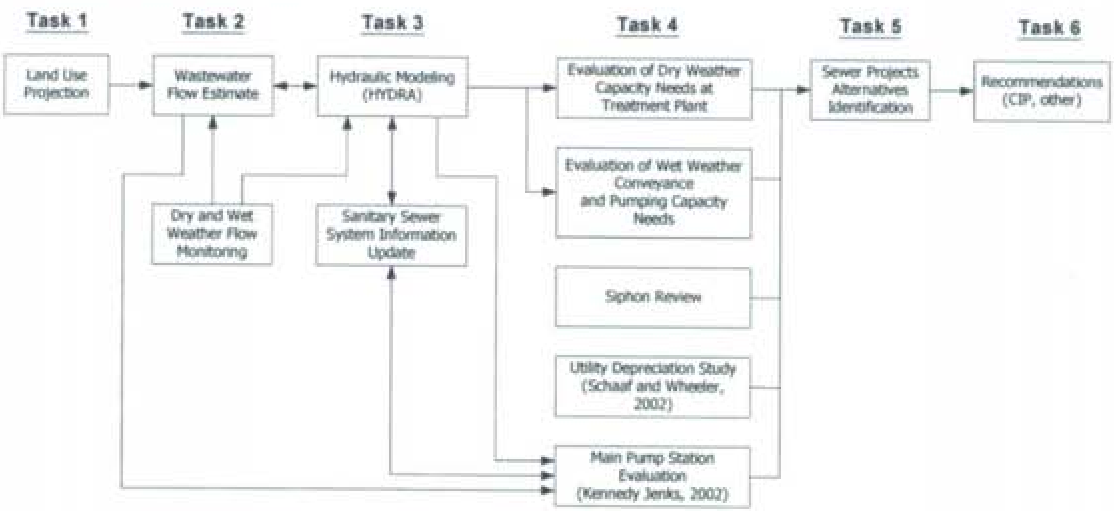
1.2 Objectives and Scope

The objectives of this Master Plan Revision are fourfold:

1. Conduct a wet weather flow monitoring program,
2. Conduct a topographical survey of the sewer system,
3. Update and calibrate the sewer system computer model under HYDRA Version 6.0, and,
4. Update the potential wet weather conveyance and pumping capacity deficiencies and associated 2002 Capital Improvement Program under existing (as of March 2004), near- (2008) and long- (2018) term conditions using the information from the wet weather flow monitoring and topographic survey data.

To achieve these objectives, the scope of work was divided into six tasks as shown in Figure 1-3. Tasks 1 through 6 are discussed in Chapter 2 through 7 of this report, respectively.

Figure 1-3: Master Plan Flowchart



1.3 Previous Studies

The City had previously prepared and updated master plans for the sanitary sewer system in 1970, 1984, 1994, and 2002. The City also completed two planning studies pertinent to this Master Plan: 1) Main Pump Station Evaluation (2002) and 2) Utility Depreciation Study (2002).

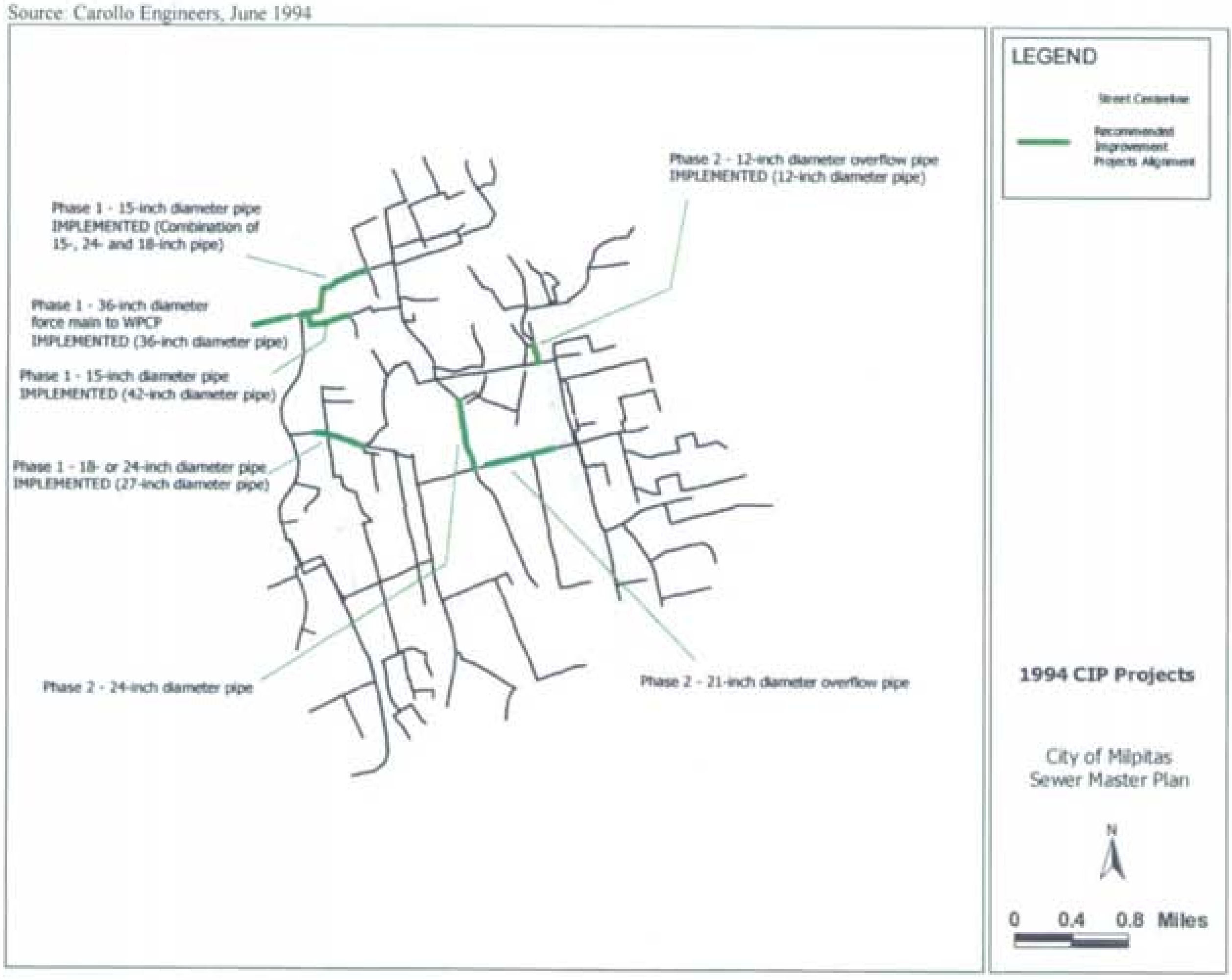
1.3.1 Sewer Master Plans

The most recent planning effort was conducted by RMC in 2003, which produced the 2002 Master Plan. This Plan provided the most up-to-date information required for the City's planning and financial efforts towards their Capital Improvement Program. The 2002 Master Plan was a comprehensive update and re-evaluation of the 1994 Sewer Master Plan Update (Carollo Engineers, 1994).

The master planning effort conducted by Carollo Engineers in 1994 considered all previous master plan efforts and developed a new capital improvement program to accommodate the City's future land use development plans to the year 2010. As part of this effort, a computer model of the City's sanitary system was developed using the hydraulic model SANSYS. The computer model was used to evaluate the sewer system improvement needs. Figure 1-4 shows the location of the 1994 Sewer Master Plan recommended projects and provides a brief description of each project.

Figure 1-4: 1994 Sewer Master Plan Recommended Projects

Source: Carollo Engineers, June 1994



One of several objectives of the 2002 Master Plan (RMC, 2003) was to update the sewer system model after its conversion by the City from SANSYS to HYDRA Version 6.0. It was recognized that the converted model needed to be reviewed, updated, and calibrated. Other planning issues reflected in the model and addressed in the 2002 Master Plan were new land use development patterns (through year 2018), potential wastewater pump capacity deficiencies, and existing siphon maintenance problems. Figure 1-5 illustrates the improvement projects recommended by the 2002 Master Plan.

Figure 1-5: 2002 Sewer Master Plan Recommended Capital Improvement Projects

1. Immediate: FY 03/04; Near-Term: FY 04/05 - FY 07/08; Mid-Term: FY 07/08 - FY 10/11; Long-Term: FY 10/11 – FY 17/1

1.3.2 Other City Studies

As part of this Master Plan Revision, the consultant team also reviewed two studies that the City recently completed to identify potential multipurpose projects.

1.3.2.1 Main Pump Station Evaluation

The Main Pump Station Evaluation – Initial Draft was completed in August 2002 (Kennedy/Jenks, 2002). The study consisted of four tasks:

- **Task 1** – Compile design and operational information on the Main PS, the existing interceptors discharging into the Main PS, and on the existing new and parallel force mains that discharge to the WPCP
- **Task 2** – Model the Main PS system as a whole (including critical interceptor system, new grinder structure, upstream interceptors) to determine how the interceptors perform under backwater conditions created by the operation of the Main PS
- **Task 3** – Investigate ways to expand the capacity of the Main PS from a base dry weather flow of 9 to 12 MGD to a peak wet weather flow of 40 MGD; and to enhance the current operation of the Main PS
- **Task 4** – Develop a control strategy that integrates wet well surface elevation, flow, and pump operation with the operation of the modulating butterfly valves installed on the new force main and added to the existing force main

Tasks 1 through 3 were specifically reviewed because of the interdependences between this study and the Sewer Master Plan Revision that include modeling the existing interceptors discharging into the Main PS, setting the water elevation at the Main PS as the downstream conditions of the sewer system model, and updating the base dry weather flow and peak wet weather flow estimates under existing, near- and long- term land use scenarios. This is discussed in Chapters 3 and 4.

1.3.2.2 Utility Depreciation Study

The Utility Depreciation Study was completed in June 2002 (Schaaf & Wheeler, 2002). The purpose of this study was to prepare an inventory of all components in the City's water distribution and wastewater collection systems, determine replacement costs for these components, and develop a schedule of replacement for various components. As part of this study, a replacement cost spreadsheet database was developed so that City staff would be able to update the inventory database on a regular basis and establish a long-term replacement program.

For the sanitary sewer system, the report contains complete lists of all pipelines segments and associated components (i.e. inlets, valves, lift stations), the expected remaining useful life of these items, and an estimated replacement cost in 2002 dollars. The report also includes a recommended implementation plan and financial analysis to determine how the City could fund replacement programs.

Table 1-1 on the next page, is extracted from the report and shows the estimated sanitary sewer system replacement costs through 2027.

The consultant team reviewed this study for pipelines and other expenditures that might coincide with this 2004 Master Plan Revision's recommended projects (see Chapter 7). Cost estimates developed in this study were also considered to develop cost criteria and project costs for this 2004 Master Plan Revision (see Chapter 6.)

Table 1-1: Estimated Sewer System Replacement Costs through 2027

YEARS FROM 2002	ESTIMATED PIPELINE REPLACEMENT COSTS [a] ($million)	OTHER EXPENDITURES ESTIMATED [b] ($million)	TOTAL ($million)
0 – 5	$3.4	$0.0	**$3.4**
5 – 10	$7.4	$0.0	**$7.4**
10 – 15	$0.0	$1.2	**$1.2**
15 – 20	$5.3	$0.0	**$5.3**
20 – 25	$0.5	$8.5	**$9.0**

Source: Schaaf & Wheeler, June 2002.
a. Include pipelines and manholes
b. Includes sewer lift stations elements

1.4 Report Content

This report is divided in seven chapters, as outlined below:

- CHAPTER 1 – INTRODUCTION (this introduction)
- CHAPTER 2 – LAND USE, describes the available land use databases and discusses existing and future land use data to be used in Chapter 3.
- CHAPTER 3 – WASTEWATER FLOWS, discusses the basis for sanitary flow projections for the City based on land use information from Chapter 2, and results from the dry and wet weather flow monitoring programs.
- CHAPTER 4 – HYDRAULIC MODEL UPDATE AND CALIBRATION, provides a history of the City's hydraulic model, and presents the model update and calibration process.
- CHAPTER 5 – SANITARY SEWER SYSTEM ANALYSIS, summarizes the identified dry weather capacity needs at the WPCP and wet weather conveyance and pumping needs identified using the hydraulic model developed in Chapter 4. It also discusses the results of the siphon review.
- CHAPTER 6 – SEWER PROJECT ALTERNATIVE ANALYSIS, presents the sewer project alternatives proposed to mitigate the deficiencies identified in Chapter 5.
- CHAPTER 7 – RECOMMENDATIONS, presents the recommended CIP based on the alternative analysis conducted in Chapter 6.

This report also contains ten appendices that are referenced in Chapters 2 through 7.

APPENDIX A – EXISTING AND FUTURE LAND USE ESTIMATES TM
APPENDIX B – DRY WEATHER FLOW MONITORING TM (2002)
APPENDIX C – WET WEATHER FLOW MONITORING TM (2002)
APPENDIX D – SYSTEM MODEL ACCEPTABILITY REVIEW TM
APPENDIX E – FLOW DIVERSION FIELD INVESTIGATION AND MODELING TM
APPENDIX F – HYDRAULIC MODEL CALIBRATION RESULTS
APPENDIX G – GIS AND HYDRA FILES
APPENDIX H – SEWER PROJECT SUPPORTING INFORMATION
APPENDIX I – WET WEATHER FLOW MONITORING PROGRAM (2004)
APPENDIX J – TOPOGRAPHIC SURVEY STUDY (2004)

REFERENCES

1. Raines, Melton and Carella, Inc., "2002 Sewer Master Plan", March 2003.
2. Raines, Melton and Carella, Inc., "2002 Water Master Plan", January 2003.
3. Kennedy/Jenks Consultants, "Main Pump Station Evaluation," Initial Draft, August 20, 2002.
4. Schaaf & Wheeler, "City of Milpitas Utility Depreciation Study," June 28, 2002.
5. EDAW, "Milpitas Midtown Specific Plan," Draft, August 2001.
6. Pizer, Inc., HYDRA Version 6.0 Users Manual, May 2000.
7. City of Milpitas, "Sewer System 1" = 100' Maps," Updated 1999.
8. City of Milpitas, "Resolution No. 6796 of the City Council of the City of Milpitas," December 1999.
9. Kennedy/Jenks Consultants, "Groundwater Infiltration Evaluation," November 1999.
10. City of Milpitas, Citywide Ortho-Photo 0.25-ft pixel resolution, July 2001.
11. City of Milpitas, "General Plan," December 1994, Amended June 1998.
12. Carollo Engineers, "City of Milpitas Sewer Master Plan Update," June 1994.
13. CH2MHill, "City of Milpitas Sewer System Master Plan," November 1984.
14. CH2MHill, "Intensive Flow Evaluation," November 1984.

CHAPTER 2 **LAND USE**

Chapter Synopsis: The identification of the most appropriate land use database and the evaluation of the existing land use and future land use scenarios are critical tasks when embarking on the sewer master planning process. It is the key for developing the existing and future base wastewater flow component of the cumulative wastewater flow.

This section provides a summary of the land use databases that were considered and the existing (as of June 2001) and future (years 2008 and 2018) land use estimates that were used for developing the 2004 Sewer Master Plan Revision. More details can be found in Existing and Future Land Use Estimates Technical Memorandum (TM) attached in Appendix A.

2.1 Land Use Database

The City's hydraulic model (HYDRA Version 6.0) provides the option of generating base wastewater flows per parcel and then linking each modeled manhole to its contributing parcels. This method makes it easy to modify local land use designations and evaluate the impact on base wastewater flow generation and conveyance capacity needs. Available parcel databases that could be used to develop land use maps were therefore identified and reviewed.

Three parcel databases were available for this Master Plan Revision: City Planning Division FileMaker database, County Geographical Information System (GIS) database, and City GIS draft[2] parcel database.

The City GIS draft parcel database was identified as the most appropriate database for the purpose of this Master Plan Revision although the land use information was not yet an attribute of the parcels and had to be developed (see Section 2.2). It provided a sufficiently reliable land use GIS database, consistent with the GIS databases under development, such as the sewer system. Other GIS databases that would be useful for future planning efforts, such as a customer complaint GIS database, could also be developed consistently with the land use GIS database.

The draft GIS parcel layer was used as a basis to import the land use information (including parcel acreage, land use type and characteristics) into the hydraulic model and generate base wastewater flows (see Chapter 3). It was necessary to develop the linkage between the parcel database (i.e. the parcel centroids) and the sanitary sewer system (i.e. nodes/manholes). By using the City's plats and record drawing information, the parcel centroids were linked to the nearest manhole or nodes in the model. The City's engineering staff provided input and reviewed the linkage.

For the purpose of this Master Plan Revision, using the draft GIS parcel layer was satisfactory. However, it is recommended that the City revise the linkage and update the hydraulic model once the parcel GIS layer and other relevant GIS layers (sewer system, customer complaints) are finalized.

2.2 Existing Land Use

Land use information per parcel is used in combination with unit base wastewater flow factors and diurnal flow patterns to distribute sanitary flows in the sewer system hydraulic model. Unit base wastewater flow factors are usually expressed in gallon per day per net acre or per person and vary with the type of land use.

A list of land use categories was developed to reflect existing land uses with similar wastewater flow generation characteristics. This classification is based on the General Plan land use designations and the land use

[2] A number of errors (missing APN numbers, overlapping parcels) were identified that impacted the parcel acreage. The major errors were corrected. The resulting parcel database was accurate enough for the purpose of the Sewer Master Plan Revision.

categories developed for the 1994 Master Plan. A total of 21 land use categories were identified. Table 2-1 provides the list of existing land use categories and associated densities. The densities were used to estimate the number of persons or constructed area per parcel since this information is not an attribute of the parcel database.

The following information was combined to create the existing land use database:

- **Zoning Map** - The zoning map provided by the City was overlaid on the parcel map using ArcView GIS Version 3.1 to create the land use base map. Some adjustments were required, as the zoning map did not overlay exactly on the parcel map. The land use map was then modified manually to account for the additional information listed below.
- **City of Milpitas General Plan** - The General Plan was used to identify areas where the actual land use differs from the zoning information.
- **City of Milpitas Ortho-Photo** - The 1999 citywide ortho-photo was used to identify undeveloped areas. It was also used to verify the type of land use on certain parcels as the zoning categories did not always correspond to the land use categories.
- **Planning Division Input** - The Planning Division staff identified vacant developable parcels on the June 2000 zoning map, unique land use areas, and large water users.
- **Service Start-Date Data** - Service start-date data provided by the City was used to identify parcels developed between June 2000 and June 2001.
- **Occupancy Data** - The City does not keep track of building vacancy. Water use records were used to estimate the occupancy of newly developed buildings west of I-880 as of June 2001.
- **School Population** – Schools were contacted to obtain number of students and staff, which was assigned to the "SCHL" parcels.
- **Hotel List** – The City provided a list of existing hotels and associated number of rooms, which was assigned to the "Hotel" parcels.
- **FY 00/01 Water Records** – Water use records were used to identify the large dischargers since no list of large dischargers and discharge flow data was immediately available. Large dischargers were identified by analyzing the FY 00/01 water records. Base wastewater flows for these users were developed as point sources, rather than calculated based on land use acreage and unit flow factors (see Chapter 3). Table 2-2 shows a list of the identified large dischargers, their FY 00/01 average water use, and their average winter water use. The winter average water use was defined as the average water use over the November 2000 through February 2001 period. This value was used since water use during this time of year is mostly indoors (minimal irrigation).

The resulting land use database is available in electronic format in Appendix G. Table 2-3 summarizes the existing land use acreage for each land use category calculated using parcel size information from the developed land use database, and the associated estimated residential population for June 2001 (59,390). This estimate is within one percent of the 2000 US census residential population estimate of 59,524, which is satisfactory for the purpose of this Master Plan Revision.

Table 2-1: Existing Land Use Categories and Associated Densities

LAND USE CATEGORIES	CODE	2001 DENSITIES: Residential Density (DU/acre)[a]	Population Density (Persons/DU)[a,b]	Maximum FAR[a,c]
Valley Floor Residential				
Single-Family Low	SFL	1 DU/parcel[c]	3.7	NA
Single-Family Medium	SFM	1 DU/parcel[d]	3.5	NA
Multifamily Medium	MFM	8[f]	3.4	NA
Multifamily High	MFH	16[b,f,g]	2.9[g]	NA
Mobile Home Park	MHP	10.5[h]	1.6	NA
Hillside Residential				
Single-Family Very Low	HVL	up to 0.1	3.7	NA
Single-Family Low	HL	up to 1	3.7	NA
Single-Family Medium	HM	up to 3	3.7	NA
Commercial				
Town Center	TC	up to 40	3	0.85
Retail Sub-center	RSC	NA	NA	0.35
General Commercial	CMRL	NA	NA	0.5
Professional/Administrative Offices	PAO	NA	NA	0.5
Industrial				
Industrial Park	INDP	NA	NA	0.4
Manufacturing/Warehousing	IND	NA	NA	0.5
Other				
Large Water User/Discharger	LWU	Water usage over 30,000 gpd. Includes jail, industrial users, and malls. Does not include hotels and schools.		
Large Hotel	Hotel	Large hotels (more than 80 rooms).		
Public/Semi-public	CVC	Includes churches, theaters, City Hall, fire station, police station, etc.		
Schools	SCHL	Includes school buildings and their parking lots. Excludes irrigated playing field.		
Parks/Recreation Irrigated[i]	PRKI	Includes parks, golf courses, schools playing fields, irrigated street meridians, cemeteries, playgrounds, etc.		
Open Space Non Irrigated	PRK	Includes stream banks, water supply and reservoirs.		
Undeveloped/Vacant Area	Vacant			

a. Source: City of Milpitas General Plan, December 1994, Amended June 1998; NA: Not Applicable
b. Adjusted per Census 2000 data
c. The Floor-Area Ratio (FAR) is defined as the ratio of floor area to gross acreage.
d. 7 DU/acre on average based on FY00/01 water use records
e. 11 DU/acre on average counting units on Aerial map
f. Planning Division staff input
g. Future Residential Density value is 22 DU/acre and future Persons/DU value is 2.7 per Planning Division staff
h. Based on FY00/01 water use records
i. This category was defined for the purpose of the 2002 Water Master Plan and kept intact in the 2002 Sewer Master Plan although no base wastewater flow is generated (could have been combined with PRK)

Table 2-2: Large Dischargers [a]

	STREET NAME	MANHOLE # (G_ID) [b]	FY 00/01 AVERAGE WATER USE (gpd) [c]	WINTER AVERAGE WATER USE (gpd) [c,d]
1	Abel St	1370	297,600	307,600
2	Milpitas Blvd	839	236,900	250,500
3	McCarthy Blvd	635	231,900	231,400
4	Tarob Ct	928	190,800	188,300
5	Buckeye Ct	639	175,900	174,200
6	Milpitas Blvd	836	166,500	154,400
7	Hillview Dr	847	163,500	160,600
8	Ames Ave	250	149,700	162,400
9	McCarthy Blvd	605	99,600	93,200
10	Hillview Dr	847	85,200	89,200
11	Los Coches St	836	76,800	69,700
12	Main St	1299	75,200	73,600
13	Hillview Dr	847	74,500	79,300
14	Tarob Ct	928	64,100	64,200
15	Yosemite Dr	849	46,500	44,800
16	Barber Ct	1398	41,300	43,000
17	Barber Ct	1390	31,600	31,700

Note: The names of the large dischargers are intentionally not shown

a. Large dischargers were identified based on water use records since no list of large dischargers and/or discharge flow data were immediately available

b. Refers to hydraulic model manhole numbering system

c. Source: FY 00/01 Water Records provided by the City

d. Average over November 2000-February 2001 period

Table 2-3: Existing (as of June 2001) Land Use Acreage and Associated Population by Land Use Category

LAND USE CATEGORY	CODE	ESTIMATED ACREAGE		ESTIMATED POPULATION
		Acres[a]	% of Total	
Valley Floor Residential				
Single-Family Low	SFL	1,435	23.8	35,600
Single-Family Medium	SFM	170	2.8	5,700
Multifamily Medium	MFM	215	3.6	5,700
Multifamily High	MFH	170	2.8	10,800
Mobile Home Park	MHP	55	0.9	1,000
Sub-Total		**2,045**	**33.9**	**58,800**
Hillside Residential				
Single-Family Very Low	HVL	15	0.2	10
Single-Family Low	HL	115	1.9	220
Single-Family Medium	HM	30	0.5	360
Sub-Total		**160**	**2.7**	**590**
Commercial				
Town Center	TC	65	1.1	**NA**
Retail Sub-center	RSC	60	1.0	
General Commercial	CMRL	265	4.4	
Professional/Administrative Offices	PAO	40	0.7	
Sub-Total		**430**	**7.1**	
Industrial				
Industrial Park	INDP	530	8.8	**NA**
Manufacturing/Warehousing	IND	760	12.6	
Sub-Total		**1,290**	**21.4**	
Other				
Large Water Use	LWU	270	4.5	**NA**
Large Hotel	Hotel	45	0.7	
Public/Semi-public	CVC	60	1.0	
Schools	SCHL	205	3.4	
Parks/Recreation Irrigated	PRKI	315	5.2	
Open Space Non Irrigated	PRK	365	6.0	
Undeveloped/Vacant Area	Vacant	850	14.1	
Sub-Total		**2,110**	**35.0**	
Total		**6,035**	**100**	**59,390**

a. Estimated acreage based on land use GIS database (Appendix G). Total acreage (excluding major streets, freeways and railroad) without the Hillside area per 1994/1998 General Plan is 5,812. Total estimated acreage without the Hillside area is 5,875. The 1.1% discrepancy is satisfactory for the purpose of this Master Plan Revision. The discrepancy is likely due to remaining errors in the draft GIS parcel database.

2.3 Future Land Use

The potential variability of Silicon Valley growth and its influence on the City land use policy makes it difficult to assess future land use within the City. The consultant team and City staff agreed that the most conservative of the reasonable scenarios, i.e. the scenario leading to the highest base wastewater flow generation within the shortest time period, should be considered for the purpose of this Master Plan Revision.

Meetings were held with the Planning Division of the City to discuss specific areas of future development/redevelopment, and identify reasonable scenarios. Documents, including the City of Milpitas General Plan and the Milpitas Midtown Specific Plan, were reviewed. The information on the type, phasing and timing of development and redevelopment, which was obtained through these discussions and from the reviewed documents and used to develop the future land use database, is detailed in Appendix A.

Key elements relevant to future land use are summarized below.

- **Planning Scenarios** – Two scenarios were evaluated as part of this Master Plan Revision: a near-term land use scenario (year 2008) and a long-term land use scenario (year 2018). Buildout of the Midtown Specific Plan area (scheduled to occur beyond the year 2018 timeframe) was considered as part of a sensitivity analysis.

- **Vacant Land Development Projects** – The area west of I-880 represents most of the vacant developable acreage in the Valley Floor area. It is mostly undeveloped industrial/commercial/office land. Other vacant developable parcels are scattered throughout the City. For the purpose of this Master Plan Revision, it was assumed that all vacant parcels in the Valley Floor area would be developed by 2008, except for some vacant parcels within the Midtown Specific Plan area that will be developed as the redevelopment projects occur (see discussion below on Redevelopment Projects in the Valley Floor Area).

  As far as the Hillside area is concerned and as a result of City Ordinance No. 38742, which fixed the limits of the City service area to the urban growth boundary on a 20-year horizon, only one significant change is anticipated to occur in the Hillside area before December 2018: a 28-dwelling unit project on the Murphy Ranch property. For the purpose of this Master Plan Revision, it is assumed that this project would be implemented after year 2008, since there are no existing permit applications or plans on file with the City.

- **Redevelopment Projects in the Valley Floor Area** – The Valley Floor area is entering a redevelopment era. Most of the redevelopment will occur in a special planning area established by the City, called the Midtown Specific Plan area. The Midtown Specific Plan area is shown in Figure 2-1. The proposed land uses in the Midtown Specific Plan area include new land use categories such as mixed use and multifamily very-high density. Table 2-4 summarizes the new densities as drafted by the Midtown Specific Plan Subcommittee (EDAW, August 2001) and used for the purpose of this Master Plan Revision. The implementation of the Midtown Specific Plan is scheduled to begin during 2002. The plan is anticipated to be complete around 2020. The Planning Division staff provided an estimate of the timing of development for each specific sub-area (2008, 2018 or later).

  Five redevelopment areas outside the Midtown Specific Plan area, described in Table 2-5, were identified by the Planning Division staff and incorporated in the future land use database.

To validate the assumptions in terms of residential densities and development timing, population was estimated for the near- and long-term scenarios, using minimum and maximum residential densities, and was compared to Association of Bay Area Government (ABAG) estimates. Figure 2-2 shows that the estimated population is consistent with the ABAG projections. For the purpose of this Master Plan Revision, the maximum residential densities were used for future residential development since the estimated population using the minimum densities is lower than the ABAG projection both for 2008 and 2018.

Future land use information for each planning scenario was added in the previously developed land use database, attached in Appendix G. Table 2-6 summarizes the future land use acreage by land use category.

Figure 2-1: Midtown Specific Plan Area

Table 2-4: Midtown Specific Plan Land Use Categories[a]

LAND USE CATEGORY	CODE	DESIGN DENSITIES		
		Residential Density (DU/acre)	Person/DU	Maximum FAR
Residential				
Multifamily Very High	MFVH	31-40	2.7	NA
Commercial				
Mixed Use	MXD	21-30	2.7	0.75
Overlay Districts [b]				
Multifamily Very High with TOD Overlay Zone	MFVH-TOD	41-60	2.7	NA
Mixed Use with TOD Overlay Zone	MXD-TOD	31-40	2.7	1.0
Manufacturing/Warehousing with TOD Overlay Zone	IND-TOD	NA	NA	0.4
Gateway Office Overlay Zone	CMRL-OO	NA	NA	1.5

a. Source: Milpitas Midtown Specific Plan, Draft (EDAW, August 2001); NA: Not Applicable

b. Transit Oriented Development (TOD) overlay zones are areas located within approximately a quarter-mile radius of the transit stations where special development standards (i.e. density and parking requirements) are tailored to the area's proximity to the transit stations

Table 2-5: Redevelopment Areas Outside of the Midtown Specific Plan Area [a]

REDEVELOPMENT AREA	CODE [b]	TIMING	DESIGN DENSITIES		
			Residential Density (DU/acre)	Population Density (Person/DU)	Maximum FAR
AREA 1 Industrial area east of Union Pacific Railroad between E Calaveras Blvd and Montague Expressway	IND	2008	NA	NA	0.4
AREA 2 Parcels located to the south west of the I-680/Calaveras Blvd intersection	MFH	2008	40	2.7	NA
AREA 3 Parcels located to the south east of the I-680/Calaveras Blvd intersection	MFH	2008/2018	40	2.7	NA
AREA 4 Construction site east of Union Pacific Railroad on Milpitas Blvd at Hanson Court	MFH	2018	40	2.7	NA
AREA 5 Town Center	CMRL	2018	NA	NA	1.7

a. Source: Discussion with Planning Division staff
b. Refer to Table 2-1

Figure 2-2: ABAG Projections 2002 versus Estimated Population

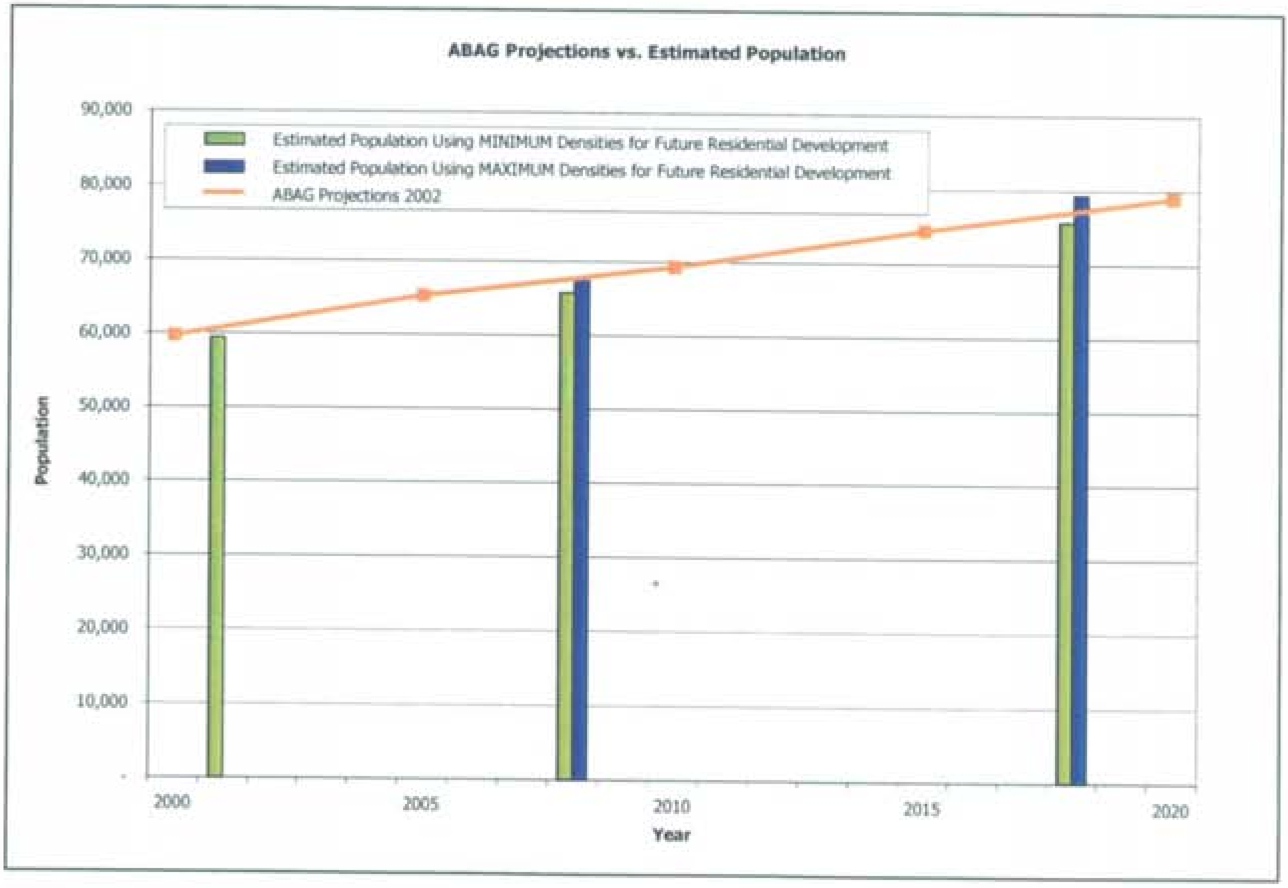


Table 2-6: Future Land Use Acreage by Land Use Category

LAND USE CATEGORY	CODE	ESTIMATED ACREAGE					
		2008		2018		Midtown Buildout	
		Acres	% of Total	Acres	% of Total	Acres	% of Total
Valley Floor Residential							
Single-Family Low	SFL	1,440	23.9	1,440	23.9	1,440	23.9
Single-Family Medium	SFM	170	2.8	170	2.8	170	2.8
Multifamily Medium	MFM	215	3.6	215	3.6	215	3.6
Multifamily High	MFH	180	3.0	195	3.2	195	3.2
Multifamily Very High	MFVH	15	0.2	50	0.8	75	1.3
Mobile Home Park	MHP	55	0.9	55	0.9	55	0.9
Sub-Total		**2,075**	**34.4**	**2,125**	**35.2**	**2,150**	**35.7**
Hillside Residential							
Single-Family Very Low	HVL	15	0.2	15	0.3	15	0.3
Single-Family Low	HL	115	1.9	115	1.9	115	1.9
Single-Family Medium	HM	30	0.5	30	0.5	30	0.5
Sub-Total		**160**	**2.6**	**160**	**2.7**	**160**	**2.7**
Commercial							
Town Center	TC	65	1.1	35	0.5	10	0.1
Retail Sub-center	RSC	65	1.1	65	1.0	60	1.0
General Commercial	CMRL	305	5.0	315	5.2	240	4.0
Professional/Administrative Offices	PAO	45	0.8	45	0.8	45	0.7
Mixed Use	MXD	--	--	10	0.1	95	1.6
Sub-Total		**480**	**8.0**	**470**	**7.6**	**450**	**7.4**
Overlay Districts [a]							
Multifamily Very High with TOD	MFVH-TOD	30	0.5	65	1.1	85	1.4
Mixed Use with TOD	MXD-TOD	15	0.3	15	0.3	35	0.6
Manufacturing/Warehousing TOD	IND-TOD	-	-	-	-	105	1.8
Gateway Office Overlay Zone	CMRL-OO	5	0.0	15	0.2	20	0.3
Sub-Total		**50**	**0.8**	**95**	**1.6**	**245**	**4.1**
Industrial							
Industrial Park	INDP	750	12.4	785	13.0	785	13.0
Manufacturing/Warehousing	IND	785	13.0	745	12.3	710	11.7
Sub-Total		**1,535**	**25.4**	**1,530**	**25.3**	**1,495**	**24.7**
Other							
Large Water Use	LWU	250	4.2	250	4.2	240	4.0
Large Hotel	Hotel	50	0.8	50	0.8	50	0.8
Parks/Recreation Irrigated	PRKI	320	5.3	320	5.3	325	5.4
Public/Semi-public	CVC	65	1.1	65	1.1	40	0.7
Schools	SCHL	205	3.4	205	3.4	205	3.4
Open Space Non Irrigated	PRK	365	6.1	365	6.1	365	6.1
Undeveloped/Vacant Area	Vacant	485	8.0	405	6.7	315	5.2
Sub-Total		**1,740**	**28.9**	**1,660**	**27.6**	**1,540**	**25.6**
Total		**6,040**	**100**	**6,040**	**100**	**6,040**	**100**

a. Transit Oriented Development (TOD) overlay zones are areas located approximately within a quarter-mile radius of the transit stations where special development standards (i.e. density and parking requirements) are tailored to the area's proximity to the transit station

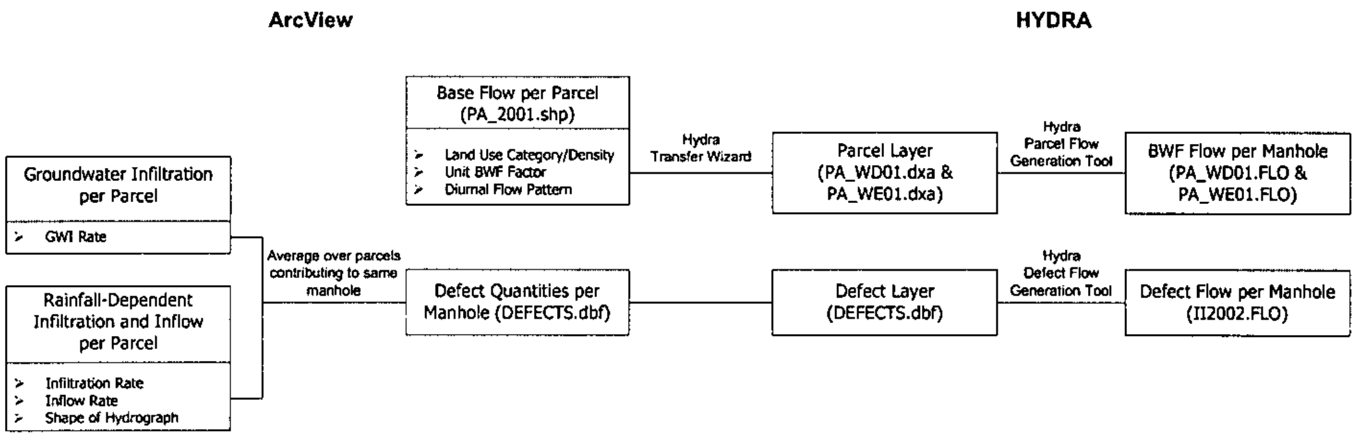
CHAPTER 3 WASTEWATER FLOWS

Chapter Synopsis: This chapter discusses the existing wastewater flows input in HYDRA for model calibration (see also Chapter 4) and the "design" wastewater flows input in HYDRA to perform the sanitary sewer system analysis under existing and future conditions (see also Chapter 5).

Wastewater flows are composed of three components: base wastewater flow (BWF), groundwater infiltration (GWI), and rainfall-dependent infiltration and inflow (RDI/I). Land use and population information per parcel is used to estimate the base wastewater flow component of the wastewater flow and was evaluated separately in Chapter 2. The GWI and RDI/I components are typically determined based on wet weather flow monitoring data.

The methodology used to generate the wastewater flow in HYDRA is shown in Figure 3-1.

Figure 3-1: Methodology for Generating Wastewater Flows in HYDRA



1. Names of electronic files (*.shp, *.dxa, *.FLO and *.dbf) correspond to flow files generated for model calibration.

3.1 Existing Flows

Each component (BWF, GWI, and RDI/I) of the existing wastewater flows that was input in HYDRA and refined until the model was calibrated (see Chapter 4) is discussed in the following paragraphs. The existing wastewater flows were then used to develop "design" wastewater flows to perform the sanitary sewer system analysis. The "design" wastewater flows are discussed in Section 3.2.

3.1.1 Base Wastewater Flow

BWF is modeled in HYDRA using the following parameters associated with each parcel:

- Unit BWF factors (in gpd/acre or gpd/person), which are combined with existing land use information (acreage or person) to calculate the average BWF
- Diurnal flow patterns, which are applied to the average BWF to model hourly flow variations

A dry weather flow monitoring program was performed for this Master Plan to provide data to update the unit BWF factors used for the City's 1994 Sewer Master Plan and establish the diurnal flow patterns associated with different land use categories. The program consisted of eight temporary flow meters installed for a two-week (three-weekend) period in July-August 2001. Areas corresponding to five distinct land use categories (SFL, SFM, CMRL, INDP, and IND) were metered. Details on the flow monitoring program (conducted by E2 Consulting Engineers) and flow data analysis are provided in the *Dry Weather Wastewater Flow Monitoring TM* in Appendix B.

Average weekday and weekend unit BWF factors for SFL, SFM, CMRL, INDP, and IND were developed based on the dry weather flow monitoring data. Unit BWF factors for the other existing land use categories identified in Table 2-1 were extrapolated from the established unit BWF factors or estimated from winter water use records provided by the City.

Typical (i.e. excluding extreme variations such as during half-time on "Super Bowl Sunday") diurnal flow patterns for SFL, SFM, CMRL, INDP, and IND were developed based on the dry weather flow monitoring data. Diurnal flow patterns for the other existing land use categories were chosen from the set of established diurnal flow patterns based on anticipated similarities in flow generation hourly variations.

The unit BWF factors and diurnal flow patterns were refined and calibrated by doing the following:

- Computing the average BWF flow for all of Milpitas and comparing it to the actual metered flow at the Main PS.
- Computing the average BWF flow generation per land use category and comparing it to the winter water use
- Running the hydraulic model and comparing calculated flow with metered flow at several locations in the system (see Chapter 4).

The calibrated unit BWF factors are summarized in Table 3-1. The calibrated weekday and weekend diurnal flow patterns are shown in Figure 3-2 and Figure 3-3, respectively. These patterns were applied to each land use category shown in Table 3-1.

A specific unit BWF factor and diurnal flow pattern was input for each parcel in the previously developed land use database. The resulting GIS shapefile (PA_2001.shp) was then exported into the hydraulic model using HYDRA's Transfer Wizard. Two HYDRA files (PA_WD01.dxa, PA_WE01.dxa) were generated. These files served to calculate the weekday and weekend BWF at each manhole (BFWD01.FLO, BFWE01.FLO) using HYDRA's built-in BWF generation tool. All the GIS and HYDRA files are attached in Appendix G. This methodology was illustrated previously in Figure 3-1.

3.1.2 Groundwater Infiltration

Groundwater infiltration (groundwater flow that enters the system consistently, 24 hours a day) is modeled in HYDRA by associating a GWI rate (in gpd/acre) with each parcel. GWI might actually vary hourly (due to tidal influence in Milpitas). However, for the purpose of this Master Plan Revision, this potential hourly fluctuation was not represented.

A wet weather flow monitoring program was undertaken for this Master Plan Revision to collect data necessary to estimate the GWI rates under saturated conditions (worse case scenario) for input in HYDRA. The flow monitoring program, which also served other purposes (see Sections 3.1.3 and Section 4.3), consisted of four temporary flow meters installed for a three-month period in December 2003–March 2004. Three additional temporary flow meters were installed for a one-month period in January–February 2004. Details on the flow monitoring program (conducted by E2 Consulting Engineers) and the flow data analysis relative to GWI rates are provided in the *Wet Weather Wastewater Flow Monitoring Program(2004)* in Appendix I.

GWI rates for areas that were not monitored were extrapolated from areas that were monitored based on similarities in location, groundwater elevation and/or age of sewer.

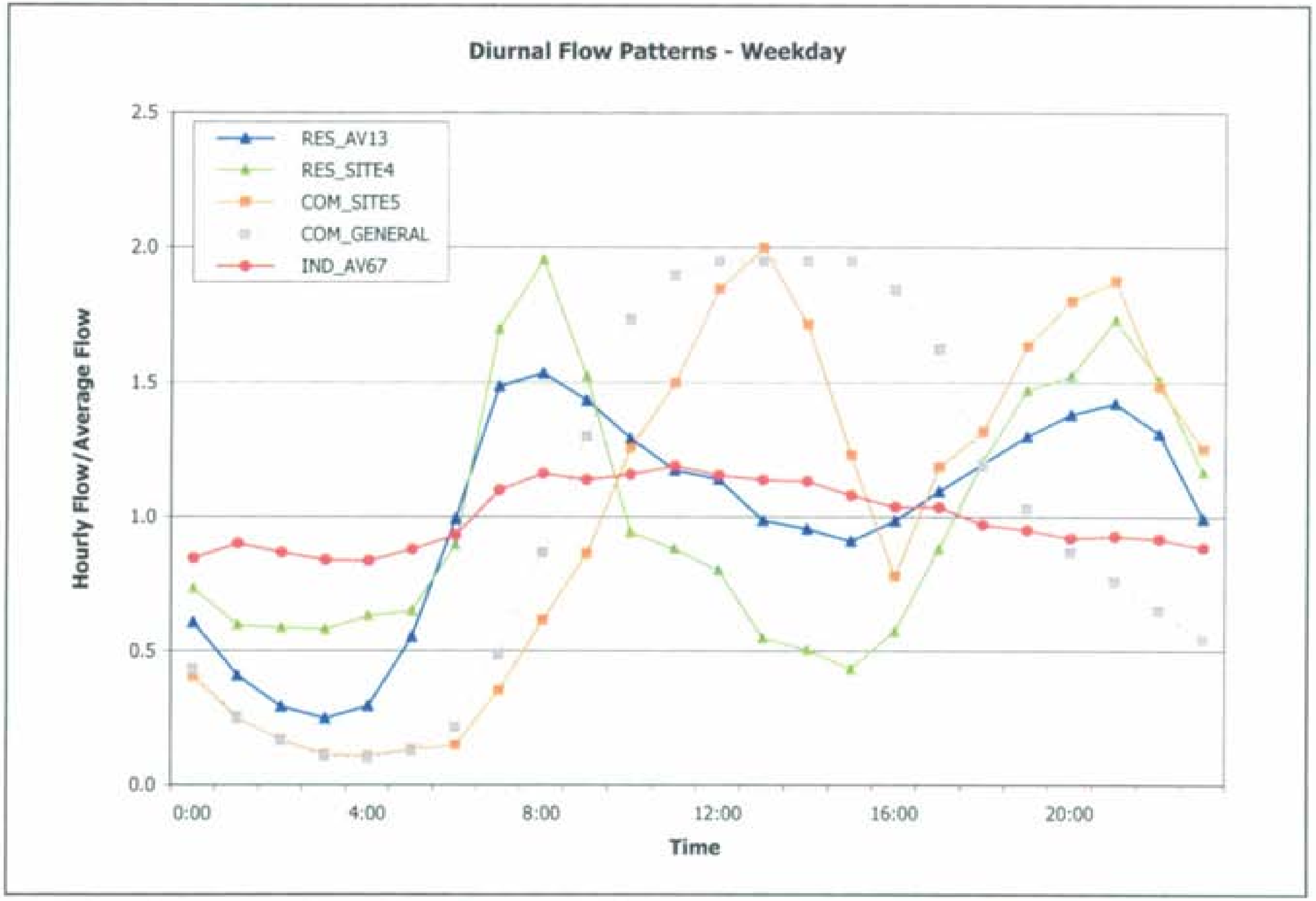
The estimated GWI rates were refined and verified by running HYDRA under existing saturated conditions with no rainfall and comparing calculated flow with the metered flow at several locations in the system (see Chapter 4). Figure 3-4 shows the established GWI rates under existing saturated conditions. The total GWI under existing saturated conditions was estimated at 1.5 MGD, or an average of 250 gpd/acre over the Valley Floor Area.

Table 3-1: Unit BWF Factor and Diurnal Flow Pattern by Land Use Category

LAND USE CATEGORY	CODE	BWF Generation per Person			BWF Generation per Acre		
		Unit BWF Factor (gpd/person)		Diurnal Flow Pattern [a]	Unit BWF Factor (gpd/acre)		Diurnal Flow Pattern [a]
		Weekday	Weekend		Weekday	Weekend	
Valley Floor Residential							
Single-Family Low	SFL	65	70	Res_av13	NA	NA	NA
Single-Family Medium	SFM	65	70	Res_av13	NA	NA	NA
Multifamily Medium	MFM	85	90	Res_site4	NA	NA	NA
Multifamily High	MFH	85	90	Res_site4	NA	NA	NA
Mobile Home Park	MHP	65	70	Res_av13	NA	NA	NA
Hillside Residential							
Single-Family Very Low	HVL	85	90	Res_site4	NA	NA	NA
Single-Family Low	HL	8	90	Res_site4	NA	NA	NA
Single-Family Medium	HM	85	90	Res_site4	NA	NA	NA
Commercial							
Town Center	TC	NA	NA	NA	1,700	1,700	Com_general
Retail Sub-center	RSC	NA	NA	NA	1,000	1,000	Com_general
General Commercial	CMRL	NA	NA	NA	1,000 [b]	1,000 [b]	Com_general [c]
Professional/Administrative	PAO	NA	NA	NA	1,000	1,000	Com_general
Industrial							
Industrial Park	INDP	NA	NA	NA	1,000 [c]	400	Ind_av67
Manufacturing/Warehousing	IND	NA	NA	NA	1,000 [d]	600 [d]	Ind_av67
Other							
Large Water Use	LWU	NA	NA	NA	Estimated from WU [e]		Ind_av67
Large Hotel	Hotel	100	100	Res_av13	NA	NA	NA
Public/Semi-public	CVC	NA	NA	NA	500	500	Com_general
Schools	SCHL	10	10	Res_av13	NA	NA	NA

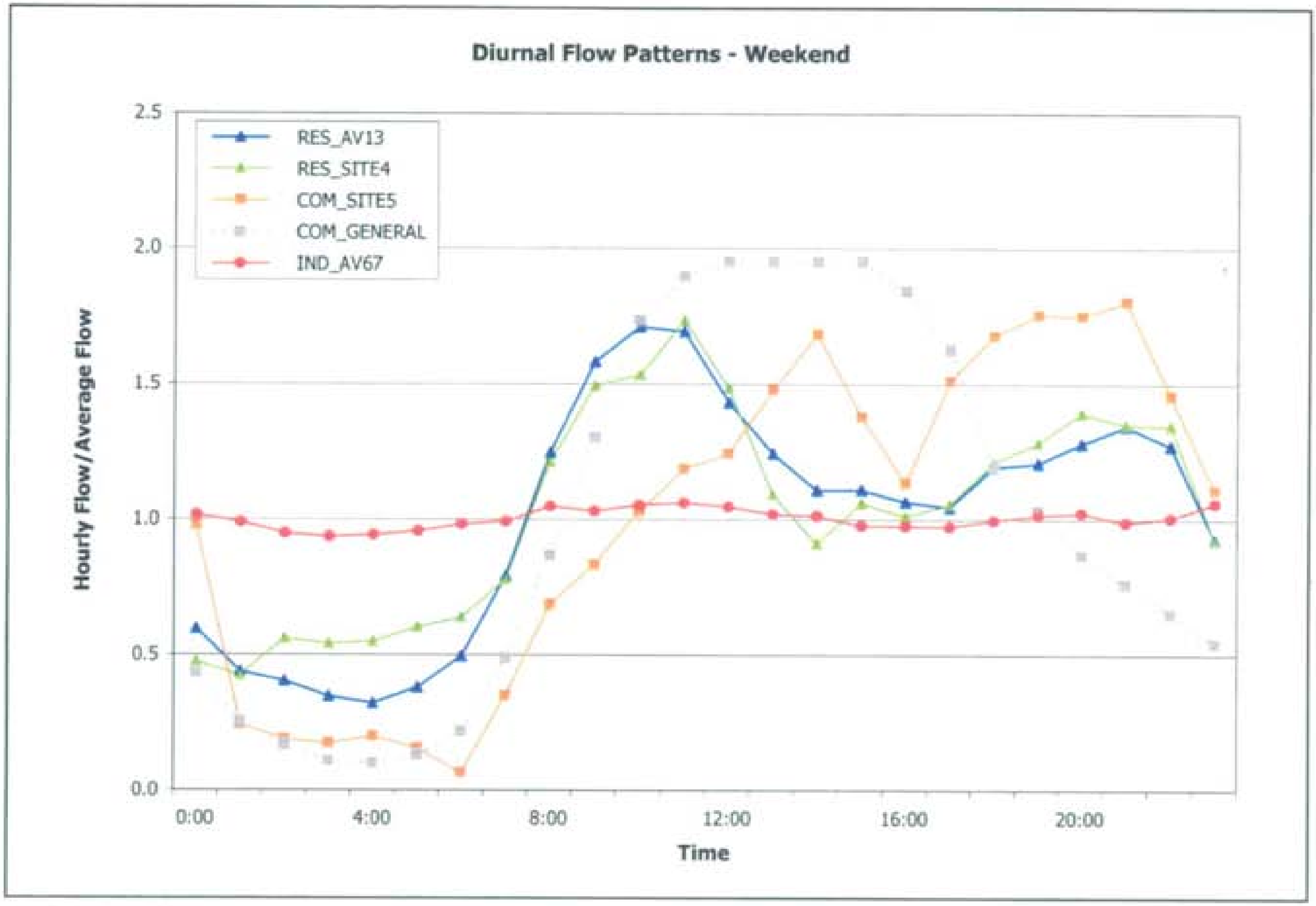
a. The nomenclature refers to the eight dry weather flow monitoring sites. The weekday and weekend diurnal flow patterns are shown in Figure 3-2 and Figure 3-3.
b. Except for McCarthy Ranch area: a unit BWF factor of 1,800 gpd/acre was assumed and Com_site5 diurnal flow pattern (see Figure 3-2 and Figure 3-3) was used.
c. Except for industrial park along McCarthy Boulevard: 1,800 gpd/acre was used for weekday.
d. Except for industrial area corresponding to dry weather flow monitoring site 7: 600 gpd/acre and 0 gpd/acre were used for weekday and weekend, respectively.
e. WU: FY00/01 winter water use. For large water users, wastewater flow was estimated as 90% of the winter water use.

Figure 3-2: Weekday Diurnal Flow Patterns



1. The diurnal flow pattern nomenclature refers to the eight dry weather flow monitoring sites (see *Dry Weather Wastewater Flow Monitoring TM* attached in Appendix B). For example RES_AV13 is the average flow pattern for residential sites 1 and 3.
2. The diurnal flow pattern for CMRL (Com_site5) was slightly modified to better represent general commercial flow generation (Com_general).

Figure 3-3: Weekend Diurnal Flow Patterns



1. The diurnal flow pattern nomenclature refers to the eight dry weather flow monitoring sites (see *Dry Weather Wastewater Flow Monitoring TM* attached in Appendix B). For example RES_AV13 is the average flow pattern for residential sites 1 and 3.
2. The diurnal flow pattern for CMRL (Com_site5) was slightly modified to better represent general commercial flow generation (Com_general).

Figure 3-4: Winter 2003/2004 GWI Rates

The established GWI rates are not entirely consistent with field observations by Public Works staff. Major infiltration problems are known to occur in the industrial park area west of I-880 and in the area bordered by Milpitas Blvd to the west, Hillview Rd to the east, Calaveras Blvd to the south and Jacklin Rd to the north (a.k.a. Hamilton area). There might be several reasons for these discrepancies:

- BWF generated on a continuous basis by industrial users and GWI are difficult to differentiate when conducting the flow data analysis. This can lead to potential overestimation of the BWF and underestimation of the GWI component (or vice-versa) in industrial areas. This could be the case for the industrial park area west of I-880.
- Much extrapolation had to be done to estimate the BWF and GWI in areas that were not monitored. Again, there might be areas where the BWF is overestimated and the GWI component is underestimated (or vice-versa). This could be the case for Hamilton area.
- BWF generated from 2001 flow monitoring data could be conservative in 2004 as a result of the economic downturn after 2001, causing the GWI values to be underestimated when calibrated with 2004 flow data. The BWF were not adjusted since they were calibrated in 2002 and it is assumed that the economy will recover in the future.

For the purpose of this Master Plan Revision, the level of accuracy is considered satisfactory since the sewer system analysis will be based on the sum of the flow components. If the City were to assess the potential problem of groundwater infiltration, it would need to devote more resources for a full investigation.

A GWI rate was input for each parcel in the parcel database. The resulting GIS shapefile (Winter 2004 gwi run 5.shp) was used to calculate the average GWI rate associated with the sewered area of each manhole (DEFECTS.dbf). The estimated GWI rates formed the basis to generate the groundwater flow at each manhole of the sewer system (DEFECTS5.flo) using HYDRA built-in defect flow generation tool. All the GIS and HYDRA files are attached in Appendix G. This methodology was illustrated previously in Figure 3-1.

3.1.3 Rainfall-Dependent Infiltration and Inflow

The RDI/I component is modeled in HYDRA by associating the following parameters to the parcels contributing to a manhole:

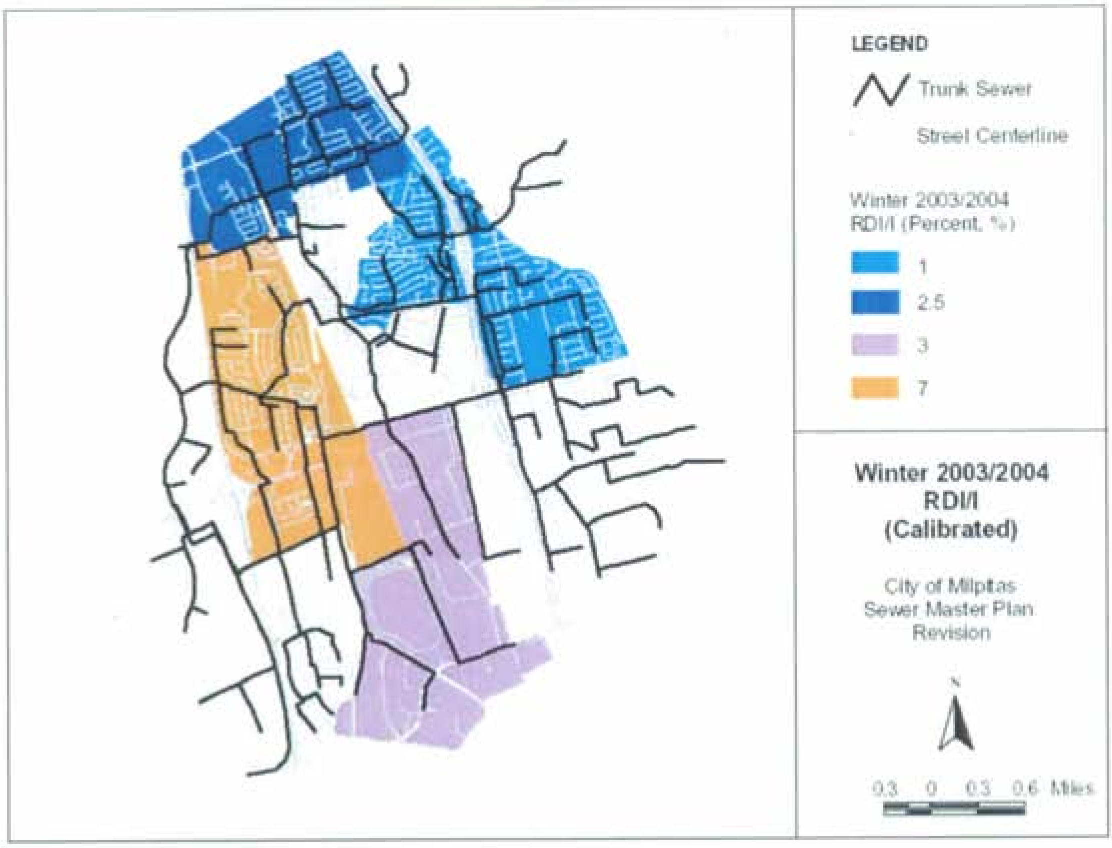
- Hyetograph (rainfall data)
- Average RDI/I rate (expressed as a percent of the total rainfall volume)
- Basic shape of hydrograph

A wet weather flow monitoring program was conducted for this Master Plan Revision to provide data necessary to estimate RDI/I rates and establish the basis of the hydrograph for input in HYDRA. As mentioned in Section 3.1.2, the wet weather flow monitoring program consisted of four temporary flow meters installed for a three-month period in December 2003–March 2004. Three additional temporary flow meters were installed for a one-month period in January–February 2004.

The data necessary to estimate RDI/I rates and the resulting hydrograph for representative sewer basins were collected during the wet weather flow monitoring period. The RDI/I rates were calibrated with downstream flow data. Figure 3-5 illustrates the established RDI/I rates under existing conditions. Due to the lack of significant rainfall during the 2002 monitoring period, the data necessary to estimate RDI/I components under saturated conditions could not be collected. Therefore a uniform RDI/I rate (5%) was used, a standard hydrograph shape was input to calibrate the RDI/I rate and a sensitivity analysis of system deficiencies for range of design RDI/I rates, as detailed in Appendix C. During the 2003/2004 flow monitoring period there was sufficient rainfall to allow for the collection of the date necessary to estimate the RDI/I components under saturated conditions. Appendix I shows the estimated RDI/I values and hydrograph shape for each metered areas for significant storm events during the flow monitoring period. Seven storm events with rain total ranging from 0.3 to 1.3 inches were evaluated. The estimated average RDI/I values, based on responses measured, ranged from 0.6 to 4.0 percent for various areas throughout the City.

An RDI/I rate was input for each parcel in the parcel database. The resulting GIS shapefile (2004 calibrated rdii (run 5).shp) was used to calculate the average RDI/I rate and hydrograph associated with the sewered area of each manhole (DEFECTS.dbf). The estimated RDI/I rates and hydrograph formed the basis to generate the rainfall-dependent infiltration and inflow volume at each manhole of the sewer system (DEFECT10.FLO) using HYDRA built-in defect flow generation tool. All the GIS and HYDRA files are attached in Appendix G. This methodology was illustrated previously in Figure 3-1.

Figure 3-5: Winter 2003/2004 Existing RDI/I Rates



3.2 Design Flows for System Analysis

Sewer system facilities are generally sized for the highest flow to be conveyed during their design life. These flow rates are termed "design flows." Each of the flow components (BWF, GWI and RDI/I) and the basis for the design flow criteria are discussed in the following paragraphs.

3.2.1 Design Base Wastewater Flow

Design base wastewater flows were developed for each scenario established in Chapter 2 (i.e. existing, near-term, and long-term).

For existing land use categories, the existing unit BWF factors and diurnal flow patterns (established in Section 3.1.1 for model calibration) were used as the "design" quantities for the existing scenario. Existing unit BWF factors and diurnal flow patterns were also used as the "design" quantities for the 2008 and 2018 land use scenarios. This is considered a reasonable assumption for the purpose of this Sewer Master Plan Revision since recent studies have shown a stable to downward trend in the Bay Area unit BWF factors, particularly for industry.

For future land use categories (Midtown Specific Plan land use categories described in Table 2-4), design BWFs were generated using the design unit BWF factors and diurnal flow patterns shown in Table 3-2 (BWFs for the "Mixed Use" land use categories were generated by combining BWF generation per person and BWF generation per acre). These unit BWF factors were extrapolated from the unit BWF factors associated with existing land use categories. The diurnal flow patterns were chosen from the set of diurnal flow patterns associated with existing land use categories based on anticipated similarities in flow generation hourly variations.

Table 3-2: Design unit BWF Factors and Diurnal Flow Patterns by Future Land Use Category

LAND USE CATEGORY	CODE	BWF Generation per Person			BWF Generation per Acre		
		Unit BWF Factor (gpd/person)		Diurnal Flow Pattern [a]	Unit BWF Factor (gpd/acre)		Diurnal Flow Pattern [a]
		Weekday	Weekend		Weekday	Weekend	
Valley Floor Residential							
Multifamily Very High	MFVH	85	90	Res_site4	NA	NA	NA
Commercial							
Mixed Use	MXD	65	70	Res_av13	1,500	1,500	Com_general
TOD Overlay Districts [b]							
Multifamily Very High with TOD	MFVH-TOD	85	90	Res_site4	NA	NA	NA
Mixed Use with TOD	MXD-TOD	65	70	Res_av13	2,000	2,000	Com_general
Manufacturing/Warehousing with TOD	IND-TOD	NA	NA	NA	1,000	600	Ind_av67
Gateway Office Overlay Zone	CMRL-TOD	NA	NA	NA	3,000	3,000	Com_general

a. The nomenclature refers to the Dry Weather Flow Monitoring sites. The weekday and weekend diurnal flow patterns are shown in Figure 3-2 and Figure 3-3.

b. Transit Oriented Development Overlay Districts are areas located approximately within a quarter-mile radius of the transit stations where development standards are tailored to the area's proximity to the transit stations.

It should be noted that the design unit BWF factor of 85 gpd/person associated with MFVH and MFVH-TOD is considered a conservative number (the City's expectation being that multi-family wastewater flow generation is lower than single-family). However, it was established based on the (validated) unit BWF factor associated with MFH (see Table 3-1). In addition, a total of 160 acres should be redeveloped under these land use categories at buildout. Assuming that the 160 acres are redeveloped at the maximum design densities, using a design unit BWF factor of 85 gpd/person (versus 65 gpd/person) would result in 0.4 MGD of incremental BWF, i.e. an incremental 3-4% of the overall BWF. This was considered to be within the range of accuracy of the flow estimates and was considered when discussing the impact on the dry weather flow capacity needs at the WPCP (see Chapter 5).

3.2.2 Design Groundwater Infiltration

Design groundwater infiltration rates were developed for each scenario (2004 existing, 2008 near-term and 2018 long-term).

The GWI rates shown on Figure 3-4 were used as the design rates under existing conditions (2004). The existing rates were calibrated based on observed flow results obtained during the 2004 winter wet-weather flow monitoring.

A 3% increase in the GWI rates was assumed between existing and near-term conditions to account for pipe deterioration. In addition, a GWI rate of 100 gpd/acre was used for near-term conditions for areas where no GWI was assumed under existing conditions. A 5% increase in the GWI rates was assumed between near-term (2008) and long-term (2018) conditions. These percentage increases are relatively conservative. However, because the GWI component does not represent a large percentage of the total design flow, the percentage increase only results in an incremental 0.45 MGD in overall design flow between existing and long-term conditions.

The design GWI rates for the various scenarios are as follows:

Existing, 2004:	1.48 MGD
Near-term, 2008:	1.84 MGD
Long-term, 2018:	1.93 MGD

3.2.3 Design Rainfall-Dependent Infiltration and Inflow

The design RDI/I component is usually defined for a storm event of a specified recurrence frequency. Typically, the design storm for sanitary sewer systems ranges from 5- to 20-year recurrence. For the 1994 Master Plan, design RDI/I rates were developed based on a 10-year storm. The same storm was used for this Master Plan Revision. The design storm is defined as follows:

Return period:	10 years
Duration:	4 hours
Intensity:	0.34 inches per hour
Total rainfall:	1.36 inches

The design storm was applied uniformly over the planning area. This assumption is relatively conservative. In reality, the intensity of the storm differs from one area of the City to the other. However, for the purposes of this Master Plan Revision, this assumption was used.

As discussed in Section 3.1.3, the data necessary to estimate RDI/I rates and the resulting hydrograph for representative sewer basins were collected during the flow monitoring period and the overall RDI/I was calibrated with downstream data. A 5% increase in the RDI/I rates was assumed for the Valley Floor between existing (2004) and near-term (2008) conditions to account for pipe deterioration. In areas where no flow was observed for existing conditions, a rate of 0.5% was assumed for near-term conditions. For long-term (2018) RDI/I, flow rates for the Valley Floor were increased by 7%. In all other areas, the long-term RDI/I was increased by 0.5%. These increases were based on the observed results of the flow monitoring program and the response of the system to the rain events. Details on the flow monitoring program (conducted by E2 Consulting Engineers) and the flow data analysis relative to RDI/I rates are provided in the *Wet Weather Flow Monitoring Program (2004)* in Appendix I.

The design RDI/I rates for the various scenarios are as follows:

Existing, 2004:	4.76 MGD
Near-term, 2008:	5.45 MGD
Long-term, 2018:	5.81 MGD

3.2.4 Design Flows

Table 3-3 shows a summary of the design flow components for the various scenarios. The table shows that the system is carrying approximately 60 percent sanitary flow and 40 percent GWI and RDI/I.

Table 3-3: Wastewater Design Flows at Various Scenarios

	ESTIMATED FLOW (MGD) [a]			
	2004	**2008**	**2018**	**2018 with MIDTOWN BUILDOUT**
Design Base Wastewater Flow	8.2	9.2	10.3	10.9
Design Groundwater Infiltration	1.48	1.84	1.93	1.93
Design Rainfall-Dependent Infiltration and Inflow	4.76	5.45	5.81	5.81
Totals	**14.44**	**16.49**	**18.04**	**18.64**

3.3 Wastewater Flow Projections

Table 3-4 presents the estimated average dry weather flow (ADWF) and 10-year design event peak wet weather flow (PWWF) for the planning area for existing and future conditions.

Table 3-4: Wastewater Flow Projections at Milpitas Main Pump Station

	ESTIMATED FLOW (MGD) [a]			
	2004	**2008**	**2018**	**2018 with MIDTOWN BUILDOUT**
Average Dry Weather Flow	9.5	10.5	11.7	12.3
Peak Wet Weather Flow [b]	19.3	21.5	23.3	24.4

a. The accuracy of these numbers are estimated to be in the 5 to 10% range due to the assumptions that were made relative to unit BWF factors, GWI rates, land use densities, etc.

b. "R" values, a 10-year design storm and shape of hydrographs as defined in 3.2.3 and Appendix I; based on the results of modeling of the system after removal of hydraulic constraints in order to determine the "true" peak flows that would occur in the system if capacity restrictions were removed.

The ADWF for the entire City was estimated based on the land use information developed in Chapter 2 and the unit BWF factors developed in Chapter 3. An average dry weather GWI component was added to the average BWF estimates. It was assumed that the average existing dry weather GWI under unsaturated conditions is approximately 1.3 MGD based on the Groundwater Infiltration Evaluation (Kennedy/Jenks Consultants, October 1999). An increase of 3% and 5% in average dry weather GWI was assumed between 2004-08 and 2008-18, respectively. No additional increase was assumed between 2018 and Midtown Specific Plan area buildout.

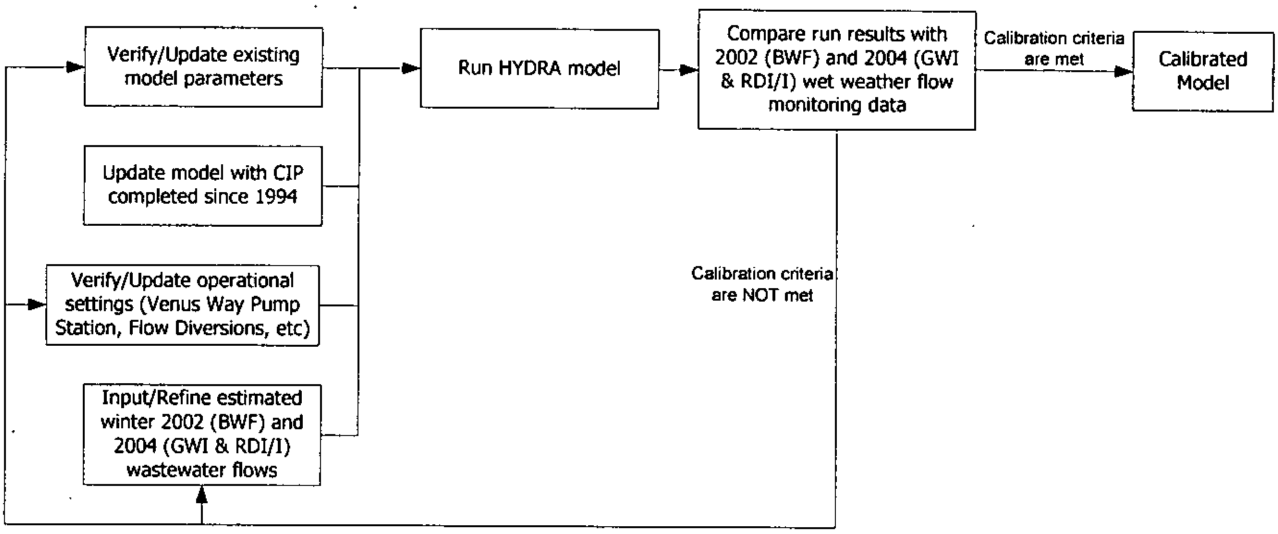
In 2002, Kennedy Jenks explored alternatives to expand the capacity of the Main Pump Station from an ADWF of nine to twelve MGD to a PWWF of 40 MGD. It should be noted that the projected PWWF used in the Main Pump Station Evaluation was higher, or more conservative, than those projected in this Master Plan Revision. The decrease in PWWF was based on 2004 flow monitoring results.

An evaluation of the flows at the Main Pump Station and the flow monitoring readings during the monitored rainfall storm events show that the average flows and the peak flows from the Main Pump Station reading and the flow meters readings are almost identical. However, the peak flows during those events were much lower than the PWWF described above (13.3-14.7 MGD versus 19.3 MGD). This leads to the conclusion that this Master Plan Revision is conservative in its estimation of peak wet weather flow.

CHAPTER 4 HYDRAULIC MODEL UPDATE AND CALIBRATION

Chapter Synopsis: This chapter presents a history of the City's HYDRA model and the updating process used to ensure that the model represents winter 2003/2004 conditions (i.e. conditions for which calibration data are available) as accurately as possible. The discussion on the model updating process also includes calibration strategy and results. The methodology used to update and calibrate the model is shown in Figure 4-1.

Figure 4-1: Methodology for Updating and Calibrating the HYDRA model



4.1 HYDRA Model History

Carollo Engineers developed a hydraulic model for the City in 1992 using the SANSYS hydraulic model software. The sanitary sewer system information was imported into the model from the existing Sewer Information Management and Maintenance System data files provided by the City at that time. Carollo Engineers updated the files to include sewer lines constructed since the 1984 plan, based on as-built drawings provided by the City. Some field verification was performed at that time, but was not documented in the 1994 Master Plan.

In 1999, West Yost & Associates converted the SANSYS model developed by Carollo Engineers to the current HYDRA Version 6.0 model. The converted model was never reviewed or calibrated. The City had already started crosschecking the integrity of the HYDRA model and recognized that there were a number of errors (such as negative slopes) in the model that needed to be corrected before the model could be used for capacity analysis purposes. In addition, the City staff was not specifically trained to use the new model.

4.2 HYDRA Model Update

As part of the 2002 Master Plan, the HYDRA model calibration process was reviewed, updated and supplemented to augment the collection system information originally imported from SANSYS into HYDRA. The imported information included the sanitary sewer system geometry and operational settings of the Venus Way Pump Station, the wetwell at the Main PS, and flow diversions within the collection system.

Table 4-1 summarizes the collection system information required for input in HYDRA to develop a model that represents winter 2004 conditions as accurately as possible. The table is organized by system elements (pipe, manhole, pump, flow diversion). It includes the source of the original data, comments on the completeness and accuracy of this data and a brief description of what was done to update and supplement this data for the purpose of this Master Plan Revision.

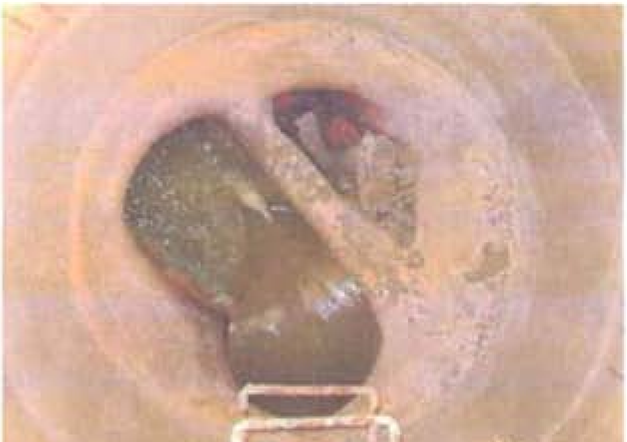
Following is a description of the key changes that were made to the HYDRA collection system layer as part of this Master Plan Revision. The updated collection system files, necessary to run HYDRA (SY_2004.dxa and project.des), are provided in Appendix G.

- **Pipe and Manhole Information as of 1999** – The City had already started crosschecking the integrity of the HYDRA model in terms of pipe and manhole information after it was imported from SANSYS. HYDRA's sewer profiles were checked for additional inaccuracies in pipe size, and rim and invert elevations (resulting in negative slopes and inverts above ground) that had not been identified/corrected. The identified inaccuracies and ways to correct them are summarized in the *Sanitary Sewer System Model Acceptability Review TM* attached in Appendix D. No surveying was conducted as most of the identified errors could be corrected based on the sewer maps. Six pipes with negative slopes are still found in the collection system. However, those were either estimated to be "real" (creek or railroad crossings) or were not expected to impact the hydraulic analysis.

  Only "abnormal" rim elevation information (ground below invert) was corrected since the recently completed rim elevation survey data was not available for a more comprehensive update. Therefore, potential errors (accuracy anticipated to be within two feet) in rim elevation should be accounted for when defining the criteria for manhole overflows. The land subsidence impact on pipe invert elevation and pipe slope was not evaluated for the purpose of this Master Plan Revision.

- **CIP completed since 1994** – The City identified only one CIP project completed since 1994 and that was not included in HYDRA's collection system layer. This project consisted of two reaches of 24-inch diameter sewer replacing 15-inch diameter sewers on Terra Mesa Way. Record drawings were used to update the model.

Figure 4-2: Example of Flow Diversion Structure[a]



a. Picture taken on August 2001 on Main Street immediately north of Hetch Hetchy Aqueduct

- **Flow Diversions** – During calibration efforts for the 2002 Sewer Master Plan, significant differences between modeled and metered flows were identified. These differences could be due to misrepresentations of the flow diversions hydraulics. In addition, existing record drawings and sewer maps did not provide all the information necessary, such as presence and height of weirs, to estimate the hydraulics of these diversions. A basic field investigation of the 11 flow diversions included in the sewer system was performed. The investigation included observation of the flow direction, measurement of invert depth and height of the weir, and photographic reconnaissance. The flow diversion located on Main Street immediately North of Hetch Hetchy is shown on Figure 4-2 as an example of the flow diversion structures included in the sewer system. The result of the flow diversion investigation (conducted by E2 Consulting Engineers in August 2002) and the estimated flow diversion hydraulics input to HYDRA are detailed in *Flow Diversion Field Investigation and Modeling TM* attached in Appendix E.

- **Topographic Survey** – Based on the CIP recommendation in the 2002 Master Plan, 36 manholes were surveyed for rim and invert elevations in 2004. Surveyed information was used to update the HYDRA collection system layer. The results of the topographic study can be found in Appendix J.

4.3 Calibration Data

In general, hydraulic models are calibrated by comparing monitored flow data to modeled hydrographs. In more sophisticated modeling (fully dynamic model), monitored flow velocity and flow depth are also compared with modeled velocity and flow depth. In addition, to verify whether the model is "realistic," historical field data such as operation issues, known deficiencies, or complaint logs can be used.

For the purpose of this Master Plan Revision, a wet weather flow monitoring program was conducted to collect downstream flow data to calibrate the model. The entire flow monitoring program, which also served other purposes (see Section 3.1.2. and Section 3.1.3), consisted of four temporary flow meters installed for a three-month period in December 2003–March 2004. Three additional temporary flow meters were installed for a one-month period in January–February 2004. Details on the flow monitoring program and flow data relative to model calibration are provided in the *Wet Weather Wastewater Flow Monitoring Program (2004)*, which can be found in Appendix I.

Operational issues, known deficiencies, and complaint logs were explored through discussions with Public Works staff and by checking the O&M database. Particular attention was paid to the past wet seasons. No area was identified as having repeated problems due to capacity issues. All the known deficiencies are related to root intrusion, debris and grease, which are operation and maintenance rather than capacity issues. Therefore, the model was calibrated based on flow monitoring data only.

4.4 Calibration Results

The model was calibrated for BWF factor (2002 Master Plan) and GWI and RDI/I (2004 Master Plan Revision) rates. The model was considered calibrated when the following criteria were met:

- The difference between the average metered and modeled flow over a 24-hour period was less than 10%, and,
- The difference between the peak hour metered and modeled flow over a 24-hour period was less than 10 to 20%.

These criteria are considered reasonable given the accuracy of the flow meters, the accuracy of the model developed based on many assumptions (estimated flows, hydraulic of the flow diversions, uniform Manning coefficient, etc.), and the incremental capacity of a pipe for a given diameter.

As shown in Figure 4-1, collection system parameters, operational settings, and winter 2003/2004 wastewater flows were adjusted until the model was considered calibrated.

The 2002 Master Plan calibration results for BWF factors and the calibration results and confirmation for GWI and RDI/I, as part of the 2004 Master Plan Revision, are presented and discussed in further detail in Appendix F.

▪

Table 4-1: Collection System Data

SYSTEM ELEMENTS	KEY PARAMETERS	SOURCE OF EXISITING HYDRA DATA [a]	COMPLETENESS AND ACCURACY OF EXISITING HYDRA DATA	HYDRA MODEL UPDATE
Pipe	Size	SY_MILBO	Known issues. Verification required.	Pipe sizes were verified as part of the *Sanitary Sewer System Model Acceptability Review TM.* The City's sewer maps served as a reference to verify pipe size. Record drawings for projects constructed since 1994 were used to input new pipe size.
	Invert Elevation (and slope) [b]	SY_MILBO	Known issues (negative slopes, invert above ground). City started correcting and revising sewer inverts. Additional verification required	Invert elevations were verified as part of the *Sanitary Sewer System Model Acceptability Review TM.* The City's sewer maps served as a reference to verify/update invert elevation information. Record drawings for projects constructed since 1994 were used to input new pipe invert elevation. Results of the detailed topographic survey (invert elevation and slope) were incorporated in the model.
	Length	SY_MILBO	No known issue. Trunk sewer alignment overlaid accurately on aerial photo.	Unchanged
	Manning's Coefficient	Project.des	Same coefficient (0.013) for all pipes	Unchanged
Manhole	Rim Elevation	SY_MILBO	Known issues (invert above ground). City started correcting and revising rim elevation. Additional verification required.	Rim elevation was verified and corrected as part of the *Model Acceptability Review TM.* Record drawings for projects constructed since 1994 were used for new manhole rim elevations. Results of rim and invert manhole elevations survey study, completed in 2004, were incorporated into the model.
	Invert Elevation	SY_MILBO	Known issues	Invert elevation was updated using updated upstream invert elevation of downstream pipe. Results of rim and invert manhole elevations survey study, completed in 2004, were incorporated into the model.
	Rim & Invert Elevation	SY_2002	Verification recommended as part of 2002 Master Plan CIP	Rim and invert elevations were surveyed and updated in HYDRA for 36 manholes.

SYSTEM ELEMENTS	KEY PARAMETERS	SOURCE OF EXISITING HYDRA DATA [a]	COMPLETENESS AND ACCURACY OF EXISITING HYDRA DATA	HYDRA MODEL UPDATE
Pumps	Number of pumps, pump capacity	Project.des	Venus Way PS modeled as one variable speed pump in a 0.1-cft wet well capable of pumping 2.3 cfs (1.5 MGD)	Updated information based on City's input: Venus Way PS consists of 2, 5-hp pumps. The operating capacity is 1.6 MGD with both pumps running There is no standby pump.
			Main PS not modeled (outlet of the system).	The water surface elevation in the Main PS wetwell set the model downstream condition. Water surface elevation documented in Main Pump Station Evaluation – Initial Draft (Kennedy Jenks, 2002) was used to set the downstream condition in HYDRA.
Diversions	Inflow vs. diverted flow	SY_MILBO	No documentation available	Revised based on *Flow Diversion Modeling TM*

a. Refers to the HYDRA files that were provided by the City at the start of this Master Plan
b. Slope is calculated from sewer inverts in HYDRA

CHAPTER 5 SEWER SYSTEM ANALYSIS

This chapter presents the results of the sewer system analysis that identified 1) dry weather capacity needs at the San Jose/Santa Clara Water Pollution Control Plant (WPCP), 2) wet weather conveyance and pumping capacity deficiencies, and 3) siphon maintenance issues and potential solutions.

5.1 Dry Weather Capacity Needs at the WPCP

Currently, all wastewater collected from the City is pumped via the Main PS to the WPCP. The WPCP has a wastewater treatment capacity of 167 MGD. The WPCP receives and treats wastewater from a total of eight cities and districts. The WPCP's treatment capacity is allocated to each discharger on the basis of the peak five-day dry weather flow, also referred to as the peak week flow. The City's current contract with the plant allows for a peak week flow capacity of 12.5 MGD. Due to the way the peak week is determined (WPCP looks at influent data to determine the five-day period of interest during the months of June through October and requests each discharger to provide their average contribution for that identified peak week), it may or may not match with Milpitas' highest dry weather flows.

The 1994 Master Plan developed dry weather capacity requirements for the City through year 2010. It estimated the peak week flow at 11.6 MGD in 2010 and concluded that no additional treatment capacity would be needed through 2010. As part of the 2002 Sewer Master Plan, the dry weather wastewater flow capacity needs at the WPCP under the near- term conditions and long-term conditions (2018 land use and 2018 land use with Midtown Specific Plan area buildout) were updated/estimated to account for the new development scenarios within the City, including the Midtown Specific Plan. No further evaluation was done as part of this Master Plan Revision. The projected peak week dry weather flows were estimated by applying a peaking factor to the average dry weather flow developed in Chapter 3, as discussed below.

5.1.1 Peak Week to Average Dry Weather Flow Ratio

The ratio between the reported peak week flow at the WPCP and the average dry weather flow at the Main PS was estimated based on reported peak week flow and dry weather flow data at Main PS available from the City. Table 5-1 shows the estimated ratio of the peak week to average dry weather flow for 1994 through 2002.

Table 5-1: Peak Week to Average Dry Weather Flow Ratio

Year	1994	1995	1996	1997	1998	1999	2000	2001	2002
Reported Peak Week Flow (MGD) [a]	7.9	7.9	8.1	9.5	8.9	8.8	10.2	9.0	8.8
Estimated Average Dry Weather Flow at Main PS (MGD) [b]	7.8	7.1	8.1	9.4	8.5	8.4	9.6	9.0	9.2
Estimated Peak Week to Average Dry Weather Flow Ratio	1.02	1.11	1.00	1.01	1.05	1.04	1.07	1.00	0.96

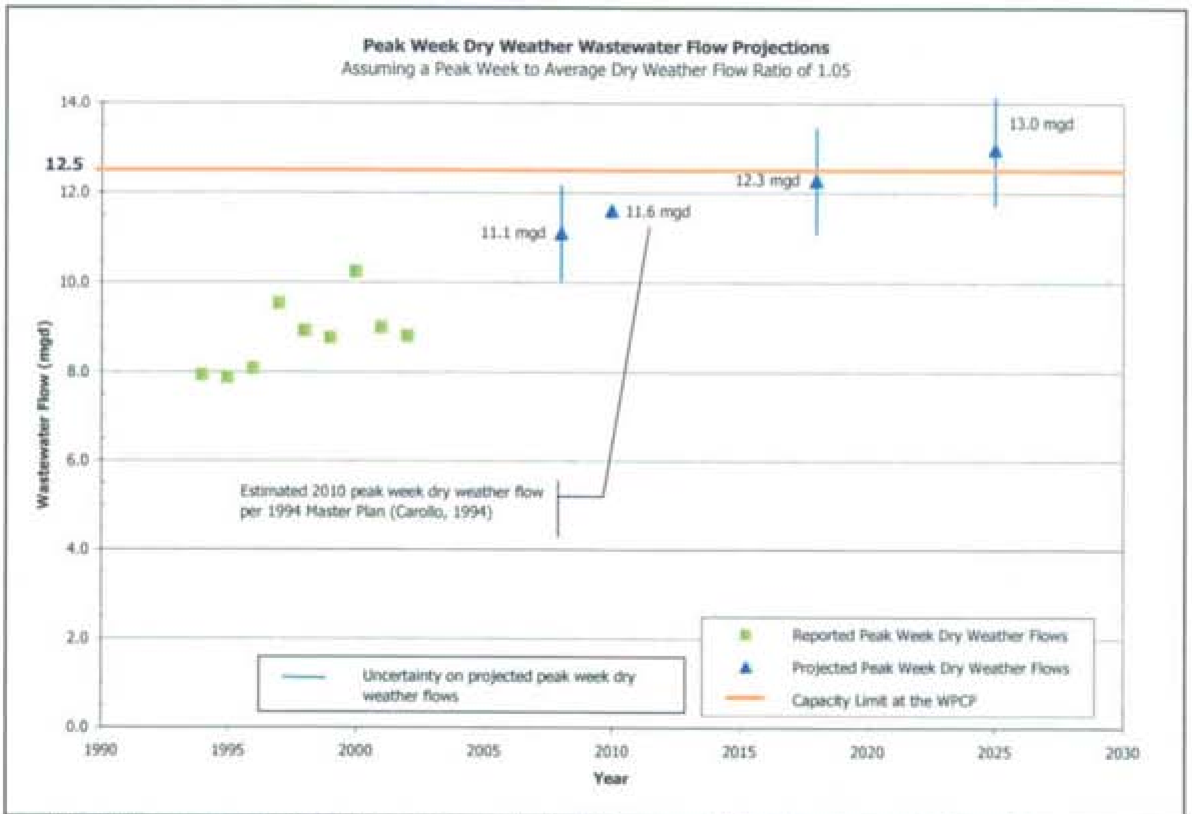
a. City of Milpitas, 1994-2002 Report to WPCP
b. Average flow between June and October based on 1994-2002 flow data at Main PS

5.1.2 Peak Week Dry Weather Wastewater Flow Projections

Figure 5-1 shows the projected peak week dry weather wastewater flows based on the average dry weather flow estimated in Chapter 3 and assuming a peak week to average dry weather flow ratio of 1.05. An uncertainty of plus or minus 10% is shown for the projected peak week dry weather wastewater flows that accounts for the uncertainties on the peak week to average dry weather flow ratio (1.05 plus 5% or minus 10% based on historical data shown in Table 5-1) and estimated average dry weather flow.

The City's capacity at WPCP of 12.5 MGD is projected to be reached around 2015. Based on the uncertainty factor, the capacity may be exceeded as soon as 2010.

Figure 5-1: Peak Week Dry Weather Wastewater Flow Projections



a. 2018 land use scenario with Midtown Specific Plan area buildout is shown as of 2025 for the purpose of this figure only.

5.2 Wet Weather Conveyance and Pumping Capacity Needs

This section presents the criteria used to determine conveyance and pumping capacity deficiencies, and the identified potential conveyance and pumping deficiencies under the existing, near- and long- term "design" flow conditions.

5.2.1 Capacity Deficiency Criteria

Table 5-2 summarizes the criteria that were used to determine conveyance and pumping capacity deficiencies.

Table 5-2: Capacity Deficiency Criteria

	CAPACITY DEFICIENCY CRITERIA
Conveyance	A pipe is considered deficient if either or both of the following condition is met at peak hour with design flows [a]: 1. There is potential for manhole overflow [b] 2. The ratio of the modeled design flow to the calculated pipe hydraulic capacity [c] exceeds 1.2 [d]
Pumping	A pump station is considered deficient if the pump station FIRM capacity [e] cannot pump calculated design flows [a] at peak hour

a. As established in Chapter 3.

b. It is assumed that there is potential for manhole overflow if the hydraulic grade line is less than 3 ft. below the ground surface. This definition accounts for potential error in rim elevation data and model accuracy. This criterion is of primary importance: a manhole overflow could represent public health risk, carries significant fines imposed by the Regional Water Quality Control Board, and could result in increase regulatory scrutiny through the pending EPA's CMOM regulations.

c. The hydraulic capacity is calculated based on the physical characteristics of the pipe and does not account for reduced capacity due to root intrusion, excessive grease accumulation, or debris. The City is responsible to ensure that 100% of the pipe capacity is available for wastewater flow.

d. This criterion was used in the 2002 Master Plan. It implies that the City allows the existing system to operate under surcharge conditions for short period of time during a 10-year storm event.

e. The City defines the FIRM capacity as the capacity with the largest pump not operating.

5.2.2 Conveyance Capacity Deficiencies

The updated HYDRA hydraulic model was run under the existing, near- and long- term design flow conditions defined in Chapter 3. Potential wet weather conveyance capacity deficiencies under each scenario were then identified based on criteria established in Table 5-2.

Figure 5-2 shows the location of identified potential wet weather conveyance capacity deficiencies for each scenario in the 2002 Sewer Master Plan and Figure 5-3 shows the location of identified potential wet weather conveyance capacity deficiencies for each scenario in this Master Plan Revision. Identified deficiencies are grouped together into sections and sequentially numbered from north to south. In all, there are ten sections where pipelines or manholes are deficient in this Master Plan Revision. Table 5-3 summarizes the deficiencies and pipe characteristics for each section. Further detail regarding each deficient section, including amount of surcharge above pipe and depth to surcharge from the ground surface, is available in Table H-1 of Appendix H. The deficiencies and pipe characteristics are further discussed for each area and section in the sections below with a comparison with deficiencies identified in the 2002 Master Plan.

5.2.2.1 Northern Area

The Northern Area has two sections, Section 1 and 2, where potential deficiencies were identified. Since the 2002 Sewer Master Plan, further data and information have removed Section 3 from the deficiency list for this 2004 Master Plan Revision.

- **Section 1** – This section is located under I-880 near the California Circle and the Main PS. The backup in this section was reduced from six pipe segments in the 2002 Master Plan to one pipe (1506/4601[3]) under I-880 with the revised model. Pipe 1506/4601 has a flat slope of 0.02 percent (compared with 0.13-4.5 percent for the same-size pipes immediately upstream and downstream). City's maintenance staff mentioned that this section was sometime backing-up due to Main PS operation. This should not be the case unless the entire interceptor sewer system is backing up, since the north interceptor sewer starts at higher elevation that the three other branches of the interceptor system. The backup is due to pipe 1506 under I-880 being a hydraulic bottleneck.
- **Section 2** – This section is located on North Milpitas Boulevard between Sunnyhills Court and Washington Drive. The model was updated with surveyed manhole data for this area and pipe 1500/14604 no longer has a flat slope issue. Hence, all manholes vulnerable to potential overflow, as identified in the 2002 Master Plan, no longer demonstrated overflow in this area in the updated model. However, even with the survey update, these three segments still show a capacity deficiency.
- **Section 3** – This section is located immediately south of the North Milpitas Boulevard and Dixon Landing Road intersection. The model was updated with surveyed manhole data for this area and no deficiency in this section was observed in the updated model.

5.2.2.2 Western Area

The Western Area has one section, Section 5A, where potential deficiencies were identified. Since the 2002 Sewer Master Plan, further data and information have removed Sections 4 and 5 from the deficiency list for this 2004 Master Plan Revision.

- **Section 4** – The 2002 Master Plan noted deficiencies in this section at Heath Street near Marylinn Drive. The model was updated with surveyed manhole data for this area and no deficiency in this section was observed in the updated model.

[3] Note that pipes are referred to using their respective G_ID/SY_NAME number from the updated HYDRA collection system layer (e.g. pipe 79/14302).

- **Section 5** – The 2002 Master Plan noted deficiencies in this section at South Abbott Avenue near West Calaveras Boulevard. The model was updated with surveyed manhole data for this area and no deficiency in this section was observed in the updated model.
- **Section 5A** – This section is located on Smithwood Street near Abbott Avenue. With the updated model, one new pipe, pipe 499/19203, was identified as deficient. This pipe has a flat slope of 0.03 percent. Since this pipe was not previously identified as deficient, surveyed data was only available for the manhole upstream of the pipe (G_ID 500/SY_NAME 19203), but not the downstream end.[4]

5.2.2.3 Central Area

The Central Area has one section, Section 6A, where potential deficiencies were identified. Since the 2002 Sewer Master Plan, further data and information have removed Sections 6 and 7 from the deficiency list for this 2004 Master Plan Revision.

- **Section 6** – This section was previously noted in the 2002 Master Plan as having capacity issues, and was located on North Milpitas Boulevard near Silverlake and Beresford Courts. The updated HYDRA analysis is showing that these pipes are not deficient.
- **Section 6A** – This section is located on South Milpitas Boulevard between Turquoise Avenue and Calaveras Boulevard. The updated GWI and RDI/I rates for this Master Plan Revision resulted in five new pipes being identified as deficient on this section of Milpitas Boulevard. Pipes 831/32607 and 835/46103 have flat slopes of 0.16 percent and 0.05 percent, respectively. The SFPUC water pipeline is located at pipe 840/46108. All pipes immediately downstream of 12-inch pipes 840/46108, 842/46401, and 844/46503 are 18-inch pipes.
- **Section 7** – This section was previously noted in the 2002 Master Plan as having capacity issues, and was located on Escuela Parkway and Angus Drive. The updated HYDRA analysis is showing that these pipes are not deficient.

[4] Surveyed data for the upstream manhole is referenced as manhole I.D 501 instead of 500.

Figure 5-2: 2002 Sewer Master Plan, Potential Wet Weather Conveyance Capacity Deficiency Locations

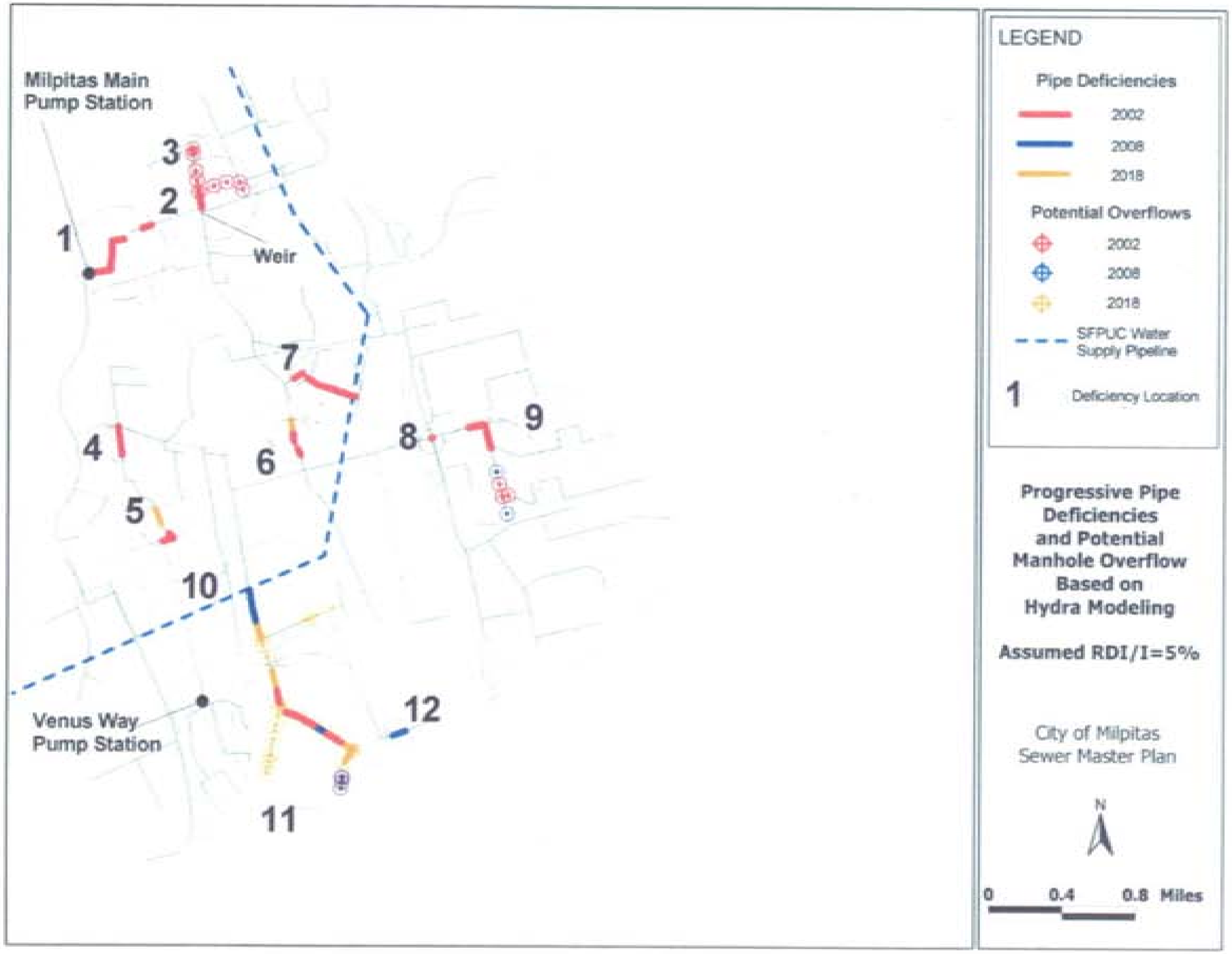


Figure 5-3: 2004 Sewer Master Plan, Potential Wet Weather Conveyance Capacity Deficiency Locations

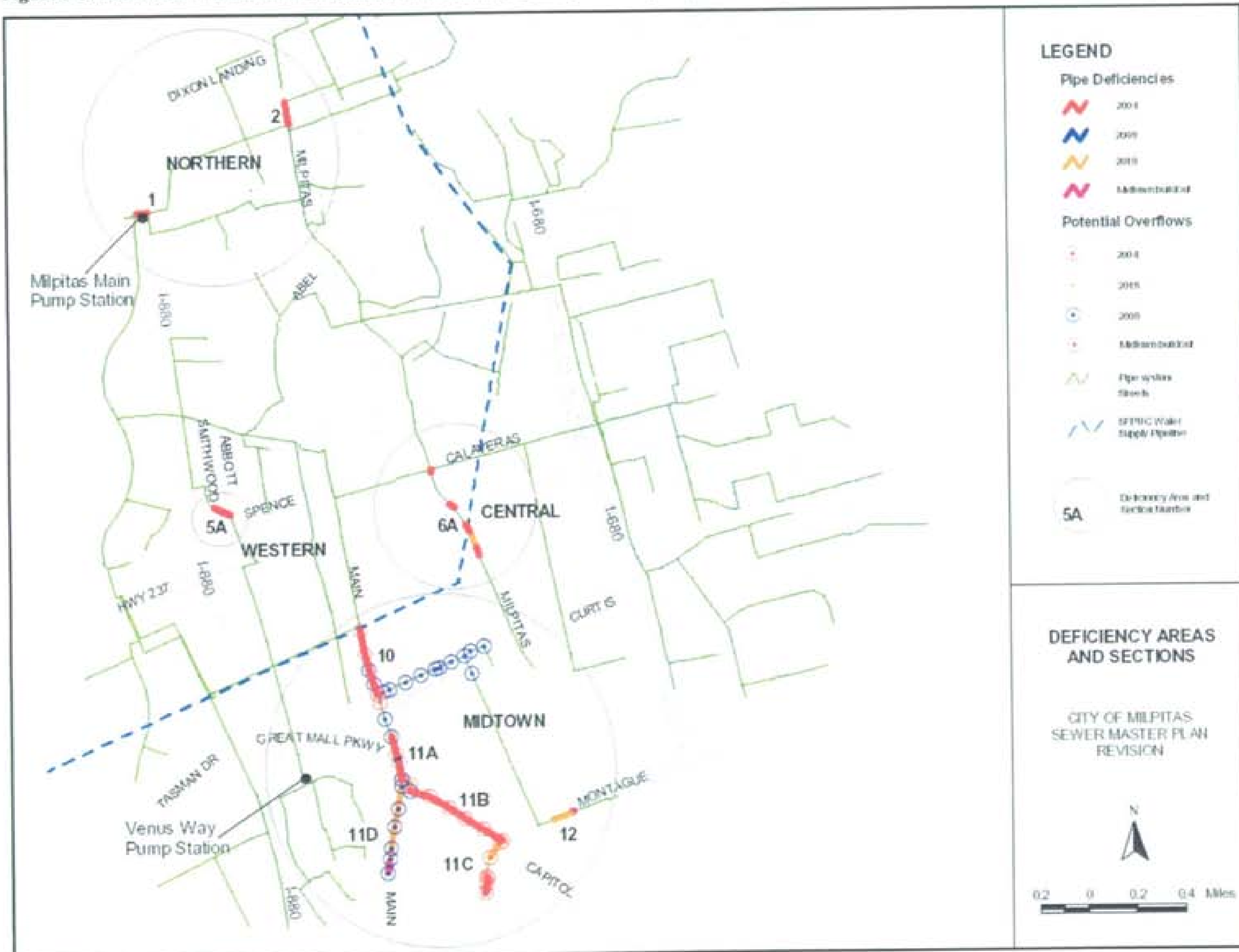


Table 5-3: Potential Wet Weather Conveyance Capacity Deficiencies [a]

SECTION	PIPE ID[b]		STREET	EXISTING DIAMETER (in.)	LENGTH (ft.)	DEPTH (ft.)	CAPACITY DEFICIENCIES[c]			
	G ID	SY NAME					2004	2008	2018	Build-out
			NORTHERN AREA							
1	1506	4601	Under I-880	18	222	14.5	2.51	2.57	2.67	2.67
	423	14603	N Milpitas Blvd between Jason and Homme	8	224	10.1	1.3	1.34	1.38	1.38
	426	14602	N Milpitas Blvd north of Homme Way	8	169	9.4	1.42	1.45	1.5	1.5
2	1500	14604	N Milpitas south of Homme Way	8	93	10.8	1.37	1.4	1.45	1.45
			WESTERN AREA							
5A	499	19203	Smithwood	15	383	8.0	1.61	1.85	1.94	1.94
			CENTRAL AREA							
	831	32607	S. Milpitas Blvd south of Calaveras	15	51	11.1	1.51	1.59	1.66	1.66
	835	46103	S. Milpitas Blvd south of Los Coches	18	132	9.3	1.44	1.49	1.58	1.58
	840	46108	S. Milpitas Blvd btwn Los Coches and Turquoise	12	152	11.3	1.71	1.77	1.86	1.86
	842	46401	S. Milpitas Blvd btwn Los Coches and Turquoise	12	420	9.7			1.21	1.21
6A	844	46503	S. Milpitas Blvd north of Turquoise	12	174	9.2	1.21	1.25	1.3	1.3
			MIDTOWN AREA							
	1292	34502	S Main St north of E Curtis Ave	18	561	9.1	1.42	1.66	2.04	2.16
	1294	34506	S Main St north of E Curtis Ave	18	339	9.1	1.51	1.77	2.17	2.29
	1296	35201	S Main St north of E Curtis Ave	18	331.4	8.4	1.29	1.51	1.86	1.96
10	1298	35205	S Main St north of E Curtis Ave	18	240.8	8.5	1.44	1.69	2.08	2.18
	258	35602	S Main St north of Great Mall Dr	18	401	11.5	1.23	1.41	2.06	2.21
	260	35603	S Main St north of Great Mall Dr	18	190	11.7	--	1.38	2.01	2.16
11A	262	36301	S Main St / Great Mall Pkwy	12	369	14.4	3.55	4.08	5.95	6.4
	908	36302	Great Mall Pkwy east of S Main	15	193	14.2			1.24	1.23
	910	36304	Great Mall Pkwy east of S Main	15	168	14.7			1.32	1.3
	913	36305	Great Mall Pkwy btwn S Main and McCandless	10	429	13.8	1.83	2.13	3.28	3.22
	915	49401	Great Mall Pkwy north of Centre Pointe Dr	10	431	14.3	1.81	2.11	3.27	3.21
	918	49101	Great Mall Pkwy / McCandless	10	495	13.1	1.84	2.14	3.33	3.27
	919	49501	Great Mall Pkwy south of Centre Pointe Dr	10	465	14.8	1.81	2.15	3.5	3.43
11B	921	49502	Great Mall Pkwy / Montague Expwy	10	451	16.4	1.29	1.54	2.61	2.56

SECTION	PIPE ID[b]		STREET	EXISTING DIAMETER (in.)	LENGTH (ft.)	DEPTH (ft.)	CAPACITY DEFICIENCIES[c]			
	G_ID	SY_NAME					2004	2008	2018	Build-out
	923	49503	Montague Expwy / E Capitol Ave	10	80	15.2			1.61	1.55
	925	49505	Montague Expwy btwn Centre Pointe and E Capitol	10	385	12.0			1.63	1.57
	927	50204	Montague Expwy / Sango Ct	8	183	5.0	1.65	1.71	2.42	2.2
	930	50203	Montague Expwy north of Sango Ct	8	143	5.7	1.78	1.85	2.62	2.38
11C	932	50202	Montague Expwy north of Sango Ct	8	28	6.0	2.64	2.73	3.87	3.51
	264	36306	S Main St south of Great Mall Pkwy	8	478	8.7			1.5	2.28
	267	36601	S Main St south of Great Mall Pkwy	8	412	8.1			1.34	2.01
	269	36602	S Main St south of Great Mall Pkwy	8	457	8.5			1.35	1.85
	271	36515	S Main St south of Great Mall Pkwy	8	234	8.3			1.35	1.75
11D	273	37204	S Main St south of Great Mall Pkwy	8	315	8.2				1.36
	275	49601	Montague Expwy west of Gladding	10	395	9.9			1.21	1.32
12	278	61108	Montague Expwy at Gladding	10	98	11.0				1.25

a. Based on HYDRA hydraulic model runs under the existing, near- and long- term design flows conditions defined in Chapter 3 – worse case scenario (weekend vs. weekday)
b. Refers to HYDRA numbering system
c. Expressed as ratio of the modeled design flow to the calculated pipe hydraulic capacity

5.2.2.4 Lower Hillside Area

Since the 2002 Sewer Master Plan, further data and information have removed Sections 8 and 9 from the deficiency list. Therefore, the Lower Hillside Area has no sections where potential deficiencies were identified in this 2004 Master Plan Revision.

- **Section 8** – This section was previously noted in the 2002 Master Plan as having capacity issues, and was located on Calaveras Boulevard at Interstate 680. The updated HYDRA analysis is showing that these pipes are not deficient.
- **Section 9** – This section was previously noted in the 2002 Master Plan as having capacity issues, and was located on Calaveras Boulevard and Carnegie Drive. The updated HYDRA analysis is showing that these pipes are not deficient.

5.2.2.5 Midtown Area

The Midtown Area remains as the area with the most pipe segments identified as having potential capacity issues. With the updated model, under the existing scenario, the number of segments with capacity issues doubled from four to eight. By the 2018 scenario with Midtown Specific Plan area buildout, the deficiency number increased from 18 segments (in the 2002 Master Plan) to 26 segments. These segments were recategorized into six sections as follows:

- **Section 10** – With the revised GWI and RDI/I rates, this section, located north of the intersection of South Main Street and East Curtis Avenue, shows capacity issues even under existing conditions. The Great Mall is loaded at a manhole downstream from this section. Due to this deficiency, a backwater effect is seen along Curtis Avenue east of South Main Street, where manhole overflows are indicated. Although these manholes demonstrate potential overflow, the existing pipes along this reach indicate no capacity issues.
- **Section 11A** – This section includes three pipe segments located on South Main Street north of Great Mall Parkway, upstream of Section 10. With the updated HYDRA model, pipe 262/36301 on South Main Street continues to be a hydraulic bottleneck for the two upstream tributary areas along Great Mall Parkway (Section 11B) and South Main Street (Section 11D). Even when Section 10 was upsized to 27-inch pipes, this section continues to be surcharged. Hence, these deficiencies are not due to downstream backwater. This bottleneck could potentially cause overflow upstream along South Main Street (pipe buried at a depth of 8 feet) under existing scenario.
- **Section 11B** – This section includes all seven pipes on Great Mall Parkway between South Main Street and Montague Expressway. With the revised GWI and RDI/I rates, five of these pipes have insufficient capacity under existing scenario. This section continues to be deficient even after Section 11A has been upsized to 27-inch pipes, indicating that these deficiencies are not due to downstream backwater.
- **Section 11C** – This section is upstream of Section 11B and includes five pipe segments on Montague Expressway between Great Mall Parkway and Sango Court. The revised GWI and RDI/I rates combined with having shallow pipes at the upstream end of the collection system could potentially cause overflow under existing conditions.
- **Section 11D** – This section consists of five pipe segments upstream of Section 11A and was previously identified in the 2002 Master Plan as being vulnerable to potential overflow due to backwater from the bottleneck created by pipe 262/36301. The pipes were not previously identified as deficient. With the revised GWI and RDI/I rates, these pipes now became surcharged starting from the 2018 scenario. The surcharged condition exists even after Section 11A has been upsized to 27-inch pipes, indicating that these deficiencies are not due to downstream backwater.

- **Section 12** – The slopes in section was updated with surveyed manhole data and pipe 275/49601 continues to have relatively flat slope averaging at 0.38 percent (compared to 1.07 percent for the same-size pipe immediately upstream and downstream). With the revised GWI and RDI/I rates, pipe 278/61108 is exhibiting marginal deficiency under by the 2018 scenario with Midtown Specific Plan area buildout.

5.2.3 PUMPING CAPACITY

In the 2002 Master Plan report, the Venus Way Pump Station[5] was identified as potentially deficient in capacity. Due to updated survey and flow information, it was determined that no pump capacity deficiencies exist.

The Venus Way Pump Station is located on Venus Way just north of West Capital Avenue. The pump station is underground, adjacent to the driveway and side yard of a private residence. In 1994, the pump station was modified from a wet well/dry well operation to a wet well with submersible pumps. It is a duplex operation, with two 5-hp pumps in the 6-foot diameter wet well; the head is approximately 15 feet. This pump station has a rated capacity of about 1.6 MGD with both pumps operating. The firm capacity of the pump station, as defined in Section 5.2.1, is approximately 0.8 MGD.

The design flow through the pump station was estimated at 0.3 MGD under the existing scenario (2004). Starting from 2008 and beyond, the design flow increases from 0.3 MGD to 0.5 MGD. Based on the criteria summarized in Section 5.2.1, the Venus Pump Station firm capacity must be at least 0.5 MGD (maximum design flow); however, the existing firm capacity of the pump station is 0.8 MGD, therefore no pump capacity deficiency exists.

5.3 Siphon Maintenance Issues and Potential Solutions

Siphon maintenance issues and potential problems were evaluated as part of the 2002 Master Plan and were not updated as part of this Master Plan Revision. Siphons are generally constructed to route flow under another utility, railroads or creeks when a standard sewer line is not feasible. The City sewer system includes a number of siphons, generally scheduled for semi-annual cleaning. The City has identified several problematic siphons (i.e. requiring cleaning weekly to quarterly), as inventoried in Table 5-4. The location of these siphons is shown on Figure 5-4.

5.3.1 POTENTIAL CAUSES

There are two main characteristics of the siphon that could contribute to maintenance problems and are discussed in the sections below:

- Siphon design criteria
- Land use upstream of the siphon

5.3.1.1 Siphon Design Criteria

Siphons are designed to flow full and under pressure to help keep them clean and free of obstacles. When the difference between dry and wet weather flows is large, the siphon usually consists of two pipelines instead of one larger pipeline. A double pipeline siphon will have flow in one pipeline meeting the minimum velocity requirements under low flow conditions, and will have flow in two pipelines under high flow conditions meting the design capacity requirement None of the siphons listed in Table 5-4 were designed as double pipelines.

Therefore, it is anticipated that at low flows the pipes will not be running full or under sufficient velocity causing accumulation of debris, grease, soils, and other raw sewage material, usually at the beginning of the rising leg of the siphon.

[5] The Main Pump Station is not evaluated as part of this Master Plan. Capacity expansion alternatives for the Main Pump Station to 40 MGD under design flow conditions were investigated as part of the Main Pump Station Evaluation (Kennedy/Jenks, 2002) – see discussion in Section 3.3.

If the siphon has a mild slope in the pipe leading to the downstream (rising) leg of the siphon, this could decrease the velocity and contribute to the buildup of sedimentation. Table 5-4 shows that the slopes of the siphon pipes range from 0.002 to 0.006, which is normal for siphon pipelines, although 0.002 is on the lower range. Another design aspect is the velocity. Siphons are usually designed to have a minimum velocity of 3 feet per second. The sewer system maps provided by the City indicate that the siphon on Main Street (SIPH03 in Table 5-4) has velocities less than 2 feet per second. Other siphons listed in Table 5-4 might also be prone to low velocities and hence low flows and sedimentation. For the purpose of this Master Plan, the velocities of the siphons were not verified because the collection system information in the current HYDRA model does not include the siphons.

5.3.1.2 Upstream Land Use

Siphons are a more prominent problem in areas where commercial or industrial customers (especially restaurants and manufacturing companies) produce large quantities of grease, fat, and wax. Special requirements for such customers, designed to help eliminate or at least decrease the amount of grease and fat entering the sewer system, are delineated in the City's sewer ordinance. One of these methods is the installation of grease traps or digesters. However, violation of the special requirements for these customers is common. Most of the problematic siphons, identified in the City, are either near a large water user, downstream of industrial or commercial areas, or next to the jail.

Figure 5-4: Location of Problematic Siphons

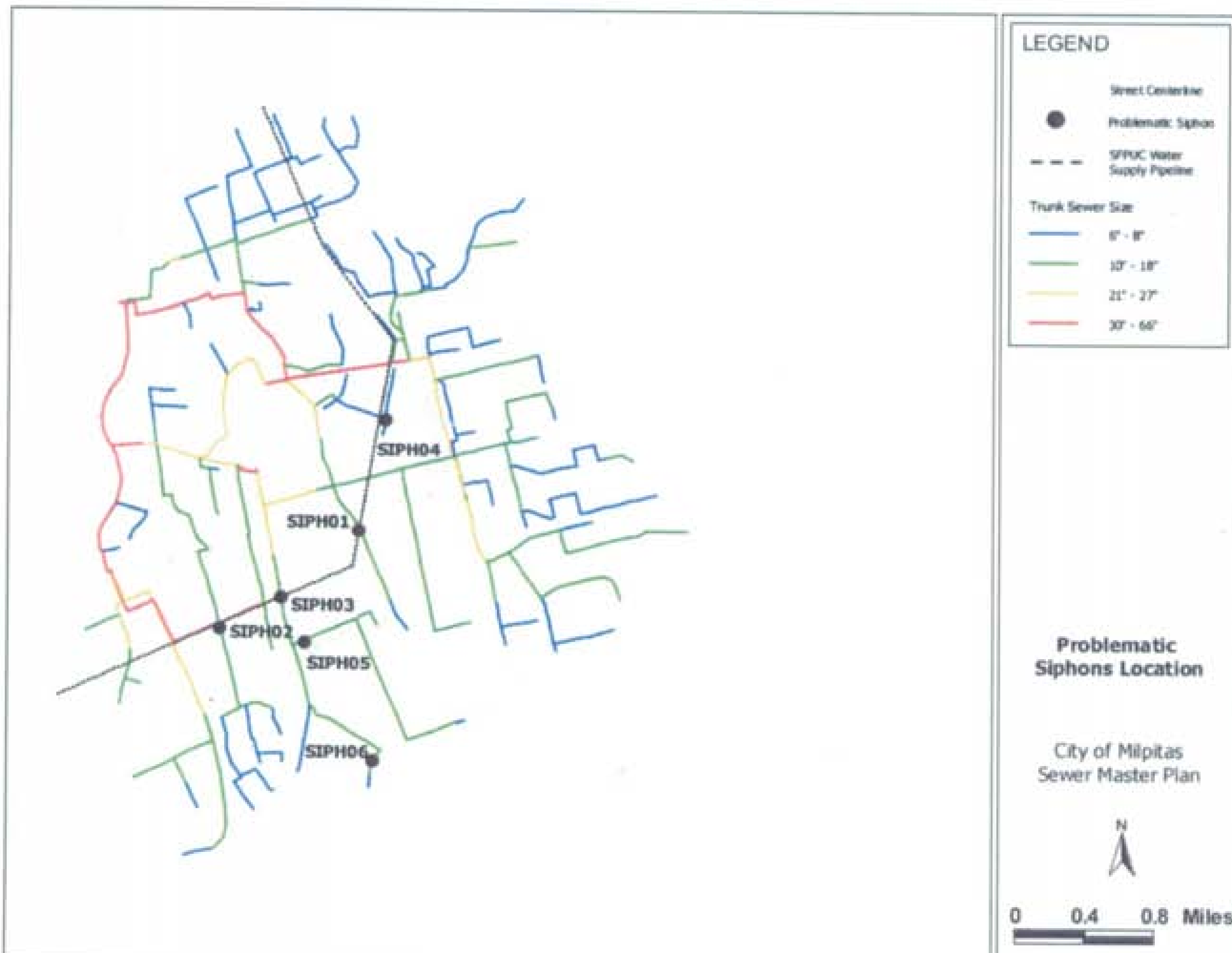


Table 5-4: Problematic Siphons Inventory

	CLEANING FREQUENCY	LOCATION	SHEET & QUADRANT #[a]	LENGTH (ft)	SIZE (in)	SLOPE	U/S INV. ELEV. (ft)	D/S INV. ELEV. (ft)	MATERIAL	CROSSING	ESTIMATED CLEANING COST[b]
SIPH01	Weekly	195 N. Milpitas Blvd	S-46/1	90	12	0.00200	6	5.7	CIP	Hetch-Hetchy Aqueduct	$98,400
SIPH02	Monthly	Elmwood at Family Golf	S-20/6	83	16	0.00602	3.79	3.29	CIP	Hetch-Hetchy Aqueduct	$22,700
SIPH03	Quarterly [c]	529 S Main St	S-34/2	50	8	0.00240	N/A	N/A	VCP	Hetch-Hetchy Aqueduct	$7,600
SIPH04	Quarterly	Gill Park between Valencia Dr and Santa Rita Dr	S-44/5	183	8	0.00246	4.27	3.82	CIP	Hetch-Hetchy Aqueduct	$7,600
SIPH05	Biannually	Curtis Av, east of Main St	S-35/2	120	18	0.00375	16.24	15.79	VCP	Railroad	$3,800
SIPH06	Biannually	400 Montague Expressway	S-50/2	129	8	0.00350	32.92	32.47	CIP	Penitencia Creek	$3,800

1. ft: feet; in: inches; U/S: Upstream; D/S: Downstream; CIPP: Cast-Iron Pipe; VCP: Vitrified Clay Pipe
2. Length, size, material, slope, and invert elevations are based on the Sewer System 1"=100' Maps provided by the City of Milpitas
3. Two problematic siphons listed on the maintenance schedule are not shown in this table: one on the southeast corner of S. Main Street at the Great Mall and the other on Santa Rita at Angus. These siphons could not be found on the plats and/or located by City personnel. For this reason, these siphons are not included in analyses.

a. Refers to Sewer System 1"=100' Maps provided by the City of Milpitas
b. 2002 net present worth cost based on 20-year life and 6% return, assuming a cleaning cost of approximately $165 per siphon, i.e. 2 hours of staff time at $60 per hour plus $45 of equipment usage (hydro-vacuum truck). Costs are based on information provided by the City of Milpitas.
c. This siphon is shown on the system maps with "V<2" to indicate that the velocity along this line is less than 2 feet per second. This condition could contribute to the need for frequent cleanings. There are actually two siphons located on this section of S. Main Street. The second siphon is located on the east side of S. Main Street. However, City maintenance staff did not indicate any cleaning problems at this location.

5.3.2 POTENTIAL SOLUTIONS

Four types of solutions were explored as part of this Master Plan:

- Lift stations
- Parallel pipelines
- Screw conveyors
- Better enforcement of discharger use of grease traps and digesters

5.3.2.1 Lift Stations

Lift stations can be used to lift flows in the downstream leg of the siphon. Lift stations have high maintenance costs and local odor production problem, making it a less than desirable solution to the siphon cleaning frequency problem. These stations come in preset packages and range between $10,000 and $50,000 including installation. The maintenance and operation of these stations can cost up to $5,000 a year. Assuming of a 20-year life, as basis for analysis, and 6% interest, the total present worth of installing and operating a lift station ranges between $70,000 and $100,000. This cost is in the range of the estimated cleaning cost for the siphon that is cleaned weekly. However, these costs do not reflect implementation issues dealing with building the lift station in the vicinity of the Hetch Hetchy aqueduct. Therefore, a lift station is not recommended for any of the problematic siphons.

5.3.2.2 Parallel Pipelines

In instances where the problem is low velocity, the construction of smaller pipeline sizes alongside already existing pipes might help. Routing wastewater through smaller pipelines would ensure increased flow and velocity, possibly reducing the frequency of cleanings. This would entail a modest modification to the manholes or possibly the installation of new structures and weirs or gates that would allow wastewater flow into the larger pipeline when the flows are high. However, the SIPH03 siphon for which low velocity is a problem runs under Hetch Hetchy Aqueduct. The cost of constructing a parallel pipeline would exceed the benefits of not having to clean the siphon. This solution is, therefore, not recommended for any of the problematic siphons.

5.3.2.3 Screw Conveyors

One system for cleaning siphons is a Screw Conveyor. The pipe screw conveyor is installed in the rising part of the siphon. Under normal operations, the blades in the middle of the screw conveyor are open allowing wastewater to pass unobstructed. To empty the siphon the screw is activated and the mixture of wastewater and any other material remaining in the siphon is lifted to the manhole. The screw conveyors could be potentially viable for some of the problematic siphons since the installation would be relatively non-disruptive. Unfortunately, this technology is common in the U.S. (several vendors were contacted but the product is only offered in Europe).

5.3.2.4 Better Enforcement of Discharger Use of Grease Traps and Digesters

The City has a sewer ordinance governing dischargers' use of grease traps and digesters and regulating the discharge of illegal materials. Increased enforcement of this mandate by dedicating more staff and resources would likely help in decreasing the frequency of the siphon cleanings. In instances when grease is clearly the issue, the City could install a grease interceptor upstream of the siphon and clean the interceptor instead of the siphon. This is the preferred solution for the problematic siphons. On the other hand, the contributing facilities could be more diligent in cleaning their grease traps or could install digesters or grinders to help with the problem. For example, the correctional facility is planning to install grinders for their facility upstream of SIPH02.

5.3.3 Conclusions

After comparing the costs of continued siphon cleaning to other solutions (e.g. lift stations, parallel pipelines, screw conveyors, grease trap/digesters), it is recommended to continue the siphon cleaning schedule. The correctional facility's installation of the grinders is an example of what facilities causing discharge problems could do to help reduce the cleaning frequency of these siphons. Should the screw conveyor technology become more common in the U.S., it could be a potential solution to be explored further. No siphon mitigation projects are included in the capital improvement program developed in Chapter 7.

CHAPTER 6 SEWER PROJECT ALTERNATIVE ANALYSIS

Chapter Synopsis: This section identifies the improvements to the sanitary sewer system based on the capacity deficiencies identified in Section 5.2 of this report. This section includes a discussion of the rationale for identifying improvements, the development of unit costs for estimating project budgets, and descriptions of the individual projects. Seven sewer conveyance capacity improvement projects but no pump station improvement projects were identified. Table 6-1 presents a summary of the project features and estimated costs.

6.1 Design Criteria

The following design criteria were used to develop the sanitary sewer and pump station improvement project alternatives to correct the potential wet-weather conveyance and pumping capacity deficiencies identified in Section 5.2 of this report.

Additional pipe capacity need was calculated (using HYDRA) as either a parallel pipe to carry flow in excess of the existing pipe design capacity or replacement (relief) pipe to carry the entire estimated flow. In order to identify the required improvements the following assumptions were made:

- Pipes that had a ratio of design flow to design capacity greater than or equal to 120 percent were included in the alternative analysis program.
- Recommended pipe sizes were based on the 2018 design flows at peak hour using a maximum allowable percent full of 90 percent for 10-inch and larger diameter pipes. Pipes impacted by the Midtown Specific Plan area redevelopment were sized based on Midtown Specific Plan area buildout conditions.
- Replacement pipes were preferred over parallel pipes because 1) the difference in the parallel and replacement pipe was generally only one or two diameters; 2) long-term maintenance is more efficient with fewer pipes and manholes in the system; and 3) underground utility congestion is minimized with fewer pipes.
- In general, short reaches of non-deficient pipes located between pipes that are deficient were included in the cost estimates to avoid downsizing of pipes in the direction of flow.

6.2 Cost Estimation Criteria

The following cost estimation criteria were used to develop planning level capital cost estimates for the identified sanitary sewer and pump station improvement projects.

6.2.1 Sanitary Sewer and Pump Costs

Sanitary sewer installation costs vary according to several factors including pipe materials, complexity of construction, traffic control, and street repair. The cost used in this Master Plan for installation of sewer pipes under "average" conditions is $16/inch-diameter/foot for 8 to 27-inch diameter. This cost includes mobilization, traffic control, trenching, dewatering, pipe installation and lateral connections, manholes, and pavement replacement. This average construction cost excludes contingency. This cost is based on sewer installation costs from a number of sources including the 1994 Milpitas Sewer Master Plan, the Newark Basin Master Plan (Union Sanitary District), the City of Milpitas Utility Depreciation Study, and the Sacramento Sewerage Facilities Expansion Master Plan. A summary of the unit costs from these projects is included in Appendix H. The unit cost factors were normalized using the May 2004 Engineering News Record Construction Cost Index (ENR-CCI) for San Francisco of 8107.

One of the potential improvement projects identified in Section 6.3 below (Project 1) would require trenchless technology since they involve Caltrans right-of-way crossing. The cost estimates for these projects include costs for jacking and boring or tunneling. The additional costs are $500 per foot of pipe plus $55,000 for jacking and receiving pits.

6.2.2 Construction Contingency and Project Implementation Multiplier

Construction contingency and project implementation multiplier were applied to each potential improvement project estimated installation cost.

A construction contingency of 30% of the estimate for pipe installation was applied to determine the construction estimate. The construction contingency is used to cover potential construction issues unforeseen at the planning level.

A project implementation multiplier was applied at a rate of 30% of the total construction cost estimate (initial estimate plus 30% contingency). The project implementation multiplier should cover:

- Administration costs
- Environmental assessments and permits
- Planning and engineering design
- Construction administration and management
- Legal fees

These assumptions are typical of planning level cost estimation.

6.3 Description of Conveyance Capacity Improvement Projects

Seven sewer conveyance capacity improvement projects have been developed for the City to correct the wet weather conveyance capacity deficiencies identified in Chapter 5. These projects and their associated costs are listed in Table 6-1. Table 6-1 presents these projects generally in order from the northern area of the City to the southern area. The priorities for implementation of the projects are discussed in Chapter 7 of this report. The total length for these projects ranges between 1.9 to 2.0 miles with an associated total estimated planning level cost of $4.7 to 5.5 millions. Detailed cost estimates are included in Appendix H of the report, along with the hydraulic grade line profiles for the deficient pipes.

Sections 6.3.1 through 6.3.12 that follow describe the projects, including the proposed pipe sizes and estimated project costs. One of these projects has alternate options, which, if prove feasible, could significantly reduce the overall improvement program costs. As a result of the new topographic and flow data used within the HYDRA model for the 2004 update, several projects originally recommended in the 2002 Master Plan have been eliminated and two new projects have been identified.

Table 6-1: Estimated Capital Cost for Sewer Conveyance Capacity Improvement Projects

PROJECT	LOCATION	LENGTH (ft)	No. of Pipe Reaches	Year of Initial Capacity Deficiency	Estimated Capital Cost, ($1000)[a]
	Northern Area				
1	I-880 Crossing	222	1	2004	450
2	N Milpitas Blvd near Jason Avenue and Homme Way	488	3	2004	130
	Western Area				
5A	Smithwood Street near Abbott Boulevard	383	1	2004	220
	Central Area				
6A	South Milpitas Boulevard between Calaveras Blvd and Turquoise	746	3	2004	490
	Midtown Specific Area				
10	Option 1 - S Main St north of E Curtis Ave	1,472	4	2004	1,080
	Option 2 - Flow diversion at Curtis Ave (Curtis Ave btwn S Main St and S Abel St)	625	1	2004	300
11A	S Main St North of Great Mall Dr	960	3	2004	700
11B	Great Mall Parkway between Montague Expressway and South Main Street	2,638	7	2004	1,250
11C	Montague Expwy between Great Mall Parkway and Sango Court	1,237	6	2004	380
11D	South Main Street south of Great Mall Parkway	2,061	6	2018	640
12	Montague Expwy west of Gladding	493	2	2018	160
	GRAND TOTAL with Option 1	10,700	36		5,500
	GRAND TOTAL with Option 2	9,853	33		4,720

a. Expressed in FY 03/04 dollars. Rounded to the nearest $10,000

6.3.1 Project No. 1-I-880 Crossing

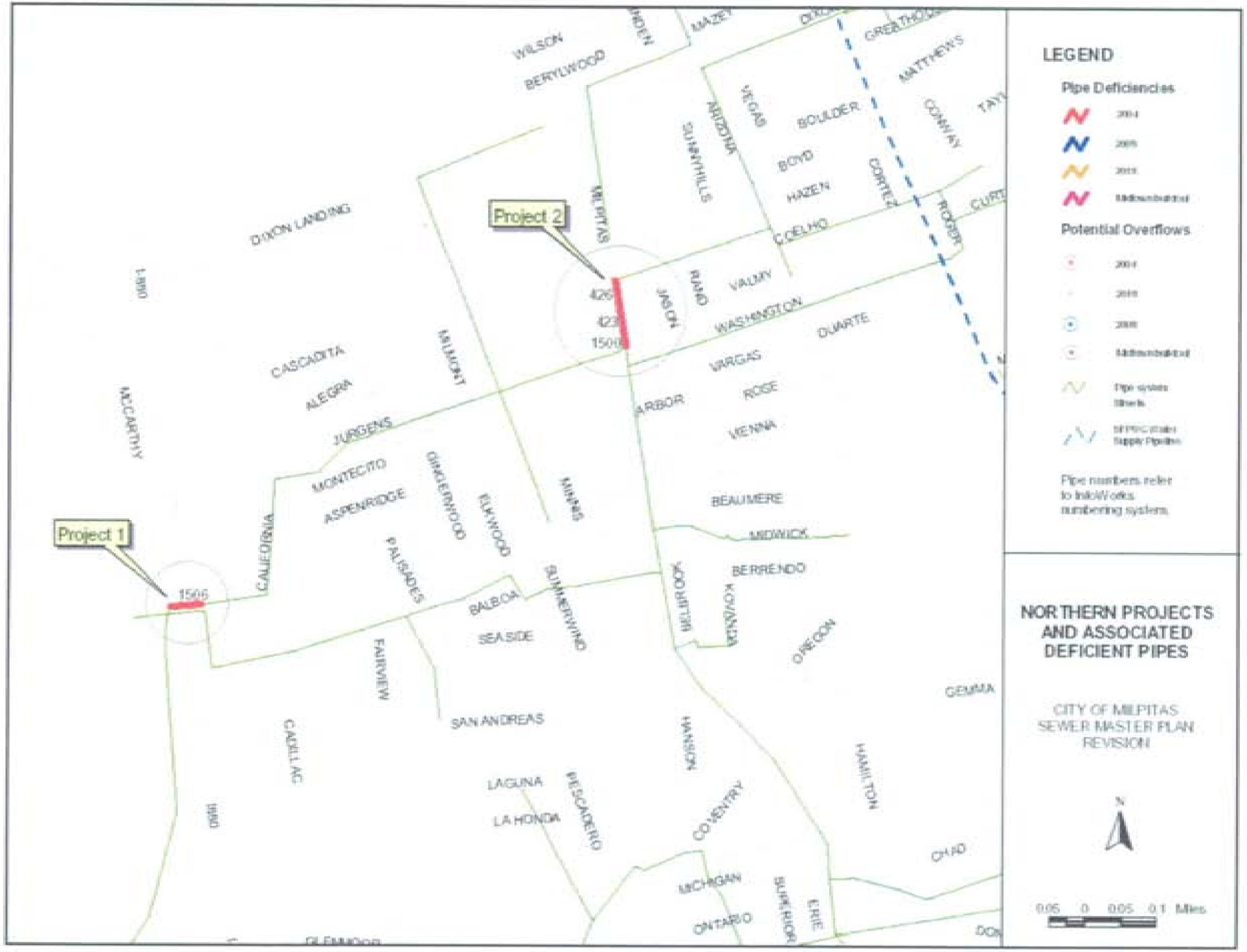
This project involves the construction of one reach of pipe across I-880, as shown on Figure 6-1. The current pipe is 18 inches in diameter and 222 feet long. The hydraulic grade line for this section, found in Appendix H, shows that surcharge is approximately four inches and it is about 13 feet deep, therefore no manhole overflows are expected. Table 6-2 lists the proposed pipe construction required for this project. The total estimated capital cost for this option is $450,000.

Table 6-2: Proposed Improvements for Project 1

PIPE ID (G_ID/ SY_NAME)[a]	DIAMETER (in)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
1506/4601	18	27	222	Replace	Pipe crosses under I-880 at N. McCarthy Rd. Pipe will exceed its 25-year life expectancy by 2010 based on the Utility Depreciation Study (Schaaf & Wheeler, 2002)

a. Refers to HYDRA numbering system

Figure 6-1: Proposed Improvements for Projects 1 and 2



6.3.2 Project No. 2-North Milpitas Boulevard near Jason Avenue and Homme Way

This project includes the construction of three reaches of pipe along North Milpitas Boulevard as shown in Figure 6-1. The three reaches flow from north to south. The existing pipes are 8 inches in diameter and total 488 feet in length. The hydraulic grade line for this section at buildout is over nine feet under ground surface, therefore no manhole overflows are expected. Table 6-3 lists the proposed pipe construction required for this project. The total estimated capital cost for this project is $130,000.

Table 6-3: Proposed Improvements for Project 2

PIPE ID (G_ID/ SY_NAME) [a]	DIAMETER (in.)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
1500/14604	8	10	95	Replace	Pipe flows north to south
423/14603	8	10	224	Replace	Pipe flows north to south
426/14602	8	10	169	Replace	Pipe flows north to south

a. Refers to HYDRA numbering system

6.3.3 Project No. 3-North Milpitas Boulevard at Dixon Landing Road

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.4 Project No. 4-Heath Street near Marylinn Drive

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.5 Project No. 5-South Abbott Avenue near West Calaveras Boulevard

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.6 Project No. 5A-Smithwood Street Near Abbot Boulevard

Project 5A was not previously identified in the 2002 Master Plan. However, the updated model and resulting hydraulic results for 2004 identified the need for this project. Project 5A includes the construction of one reach of pipe along Smithwood Street near Abbot Boulevard as shown in Figure 6-2. The existing pipe is 15 inches in diameter and 383 feet in length. The hydraulic grade line is over six feet under the ground surface; therefore no manhole overflow is expected. Table 6-4 lists the proposed pipe construction required for this project. The total estimated capital cost for this project is $220,000.

Table 6-4: Proposed Improvements for Project 5A

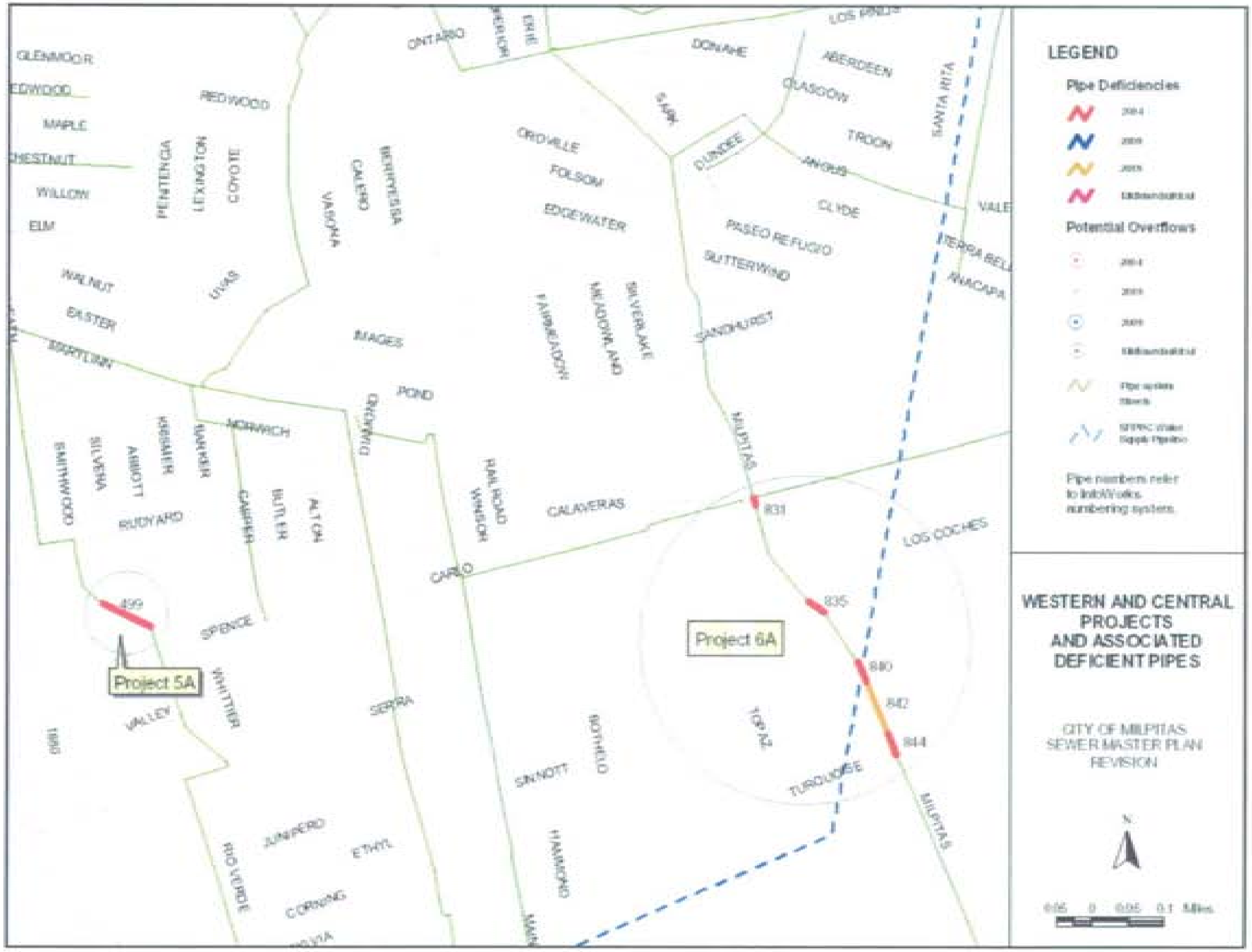
PIPE ID (G_ID/ SY_NAME) [a]	DIAMETER (in)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
499/19203	15	21	383	Replace	

a. Refers to HYDRA numbering system

6.3.7 Project No. 6-North Milpitas Boulevard near Civic Center

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

Figure 6-2: Proposed Improvements for Projects 5A and 6A



6.3.8 Project No. 6A-South Milpitas Boulevard between Calaveras Boulevard and Turquoise

Project 6A was not previously identified in the 2002 Master Plan. However, the updated model and resulting hydraulic results for 2004 identified the need for this project. Project 6A includes the construction of five reaches of pipe along South Milpitas Boulevard between Calaveras Boulevard and Turquoise as shown in Figure 6-2. The existing pipes range in size between 12 and 18 inches in diameter and total 929 feet in length. Pipes 831/32607 and 835/46103 have minimum surcharge as shown on the hydraulic grade lines for these pipelines at buildout; therefore, no improvements of these pipelines are proposed. One reach of pipe (838/46107) does not need to be replaced because it is currently the same diameter as the recommended upstream pipes. The pipelines that are proposed to be replaced are all 12 inches in diameter and they total 746 feet in length. Table 6-5 lists the proposed pipe construction required for this project. The total estimated capital cost for this project is $490,000.

Table 6-5: Proposed Improvements for Project 6A

PIPE ID (G_ID/ SY_NAME) a	DIAMETER (in)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
831/32607	15	Not needed			The surcharge is minimum even at buildout. No improvement is recommended.
835/46103	18	Not needed			The surcharge is minimum even at buildout. No improvement is recommended
838/46107	18	Not needed			Pipe not deficient and same size as recommended upstream pipes
840/46108	12	18	152	Replace	
842/46401	12	15	420	Replace	
844/46503	12	15	174	Replace	

a. Refers to HYDRA numbering system

6.3.9 Project No. 7-Escuela Parkway and Angus Drive

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.10 Project No. 8-Calaveras Boulevard at I-680

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.11 Project No. 9-Calaveras Boulevard and Carnegie Drive

In the 2002 Sewer Master Plan Update, a project was identified for this deficiency. However, further topographic data and the resulting hydraulic analyses have eliminated the need for this project.

6.3.12 Project No. 10-South Main Street North of Curtis Avenue

There are two possible options shown in Figure 6-3 for correcting the capacity deficiencies at this site. The first option is to provide additional capacity and the second is to divert a portion of the upstream flow to another sewer.

6.3.12.1 Option 1 - Relief Sewer on South Main Street

This option includes the construction of four reaches of pipe along South Main Street from the intersection of Curtis Avenue to the SFPUC water supply pipeline as shown on Figure 6-3. The current pipes are 18 inches in diameter and total 1,472 feet in length. Table 6-6 lists the proposed pipe construction required for this option. The total estimated capital cost for this option is $1,080,000.

Table 6-6: Proposed Improvements for Project 10 – Option 1

PIPE ID (G_ID/ SY_NAME) [a]	DIAMETER (in)		LENGTH (ft)	TYPE
	Existing	Recommended		
1292/34502	18	24	561	Replace
1294/34506	18	24	339	Replace
1296/35201	18	21	331	Replace
1298/35205	18	21	241	Replace

a. Refers to HYDRA numbering system

6.3.12.2 Option 2 - Flow Diversion at Curtis Avenue

This option consists in diverting flow at the intersection of North Main Street and Curtis Avenue. This option requires a new, 18-inch diameter pipe that will total 625 feet long. This pipe would have one or two manholes. It would divert a portion of the flow from the sewer on South Main Street to the sewer on South Abel Street as shown on Figure 6-3. A key design consideration at this location is access for the fire station at the intersection of South Main Street and West Curtis Avenue. The estimated capital cost for this option is $300,000.

6.3.13 Project No. 11-Great Mall Project

This project includes the construction of twenty-two reaches of pipe along South Main Street, Great Mall Parkway, and Montague Expressway as shown in Figure 6-3. The existing pipes are 8 to 18 inches in diameter and total 6,895 feet in length. This project has been divided into four phases for construction based on the recommended replacement pipes, location, and capacity deficiency time trigger. Table 6-7 lists the proposed pipe construction required for this project. The total estimated capital cost for this project is $2,970,000.

Table 6-7: Proposed Improvements for Project 11

PIPE ID (G_ID/ SY_NAME) [a]	DIAMETER (in)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
Phase A-South Main Street North of Great Mall Parkway					
258/35602	18	27	401	Replace	
260/35603	18	27	190	Replace	
262/36301	12	27	369	Replace	
Phase B-Great Mall Parkway					
908/36302	15	18	193	Replace	
910/36304	15	18	168	Replace	
913/36305	10	18	429	Replace	
918/49101	10	18	495	Replace	
915/49401	10	18	431	Replace	
919/49501	10	18	465	Replace	
921/49502	10	15	451	Replace	
Phase C-Montague Expressway					
923/49503	10	12	80	Replace	
925/49505	10	12	385	Replace	
934/50201	10	12	418	Replace	Pipe not deficient, but replacement with upstream diameter is recommended
932/50202	8	15	28	Replace	Pipe has a much milder slope than the upstream pipe and therefore requires larger pipe size.
930/50203	8	12	143	Replace	
927/50204	8	12	183	Replace	

Phase D- South Main Street South of Great Mall Parkway					
912/36303	8	12	165	Replace	Pipe not deficient, but replacement with upstream diameter is recommended
264/36306	8	12	478	Replace	
267/36601	8	12	412	Replace	
269/36602	8	12	457	Replace	
271/36515	8	10	234	Replace	
273/37204	8	10	315	Replace	

a. Refers to HYDRA numbering system

6.3.14 PROJECT NO. 12-MONTAGUE EXPRESSWAY WEST OF GLADDING COURT

This project includes construction of two reaches of pipe to correct a bottleneck on Montague Expressway at Gladding Court as shown on Figure 6-3. The existing pipes are 10 inches in diameter and 493 feet in length. The proposed pipe construction entails replacement of the existing sewer with 12-inch diameter pipe. The surcharge in these pipelines are minimum and are not observed until 2018 and buildout. Table 6-8 lists the proposed pipe construction required for this project. The total estimated capital cost for this project is $160,000.

Table 6-8: Proposed Improvements for Project 12

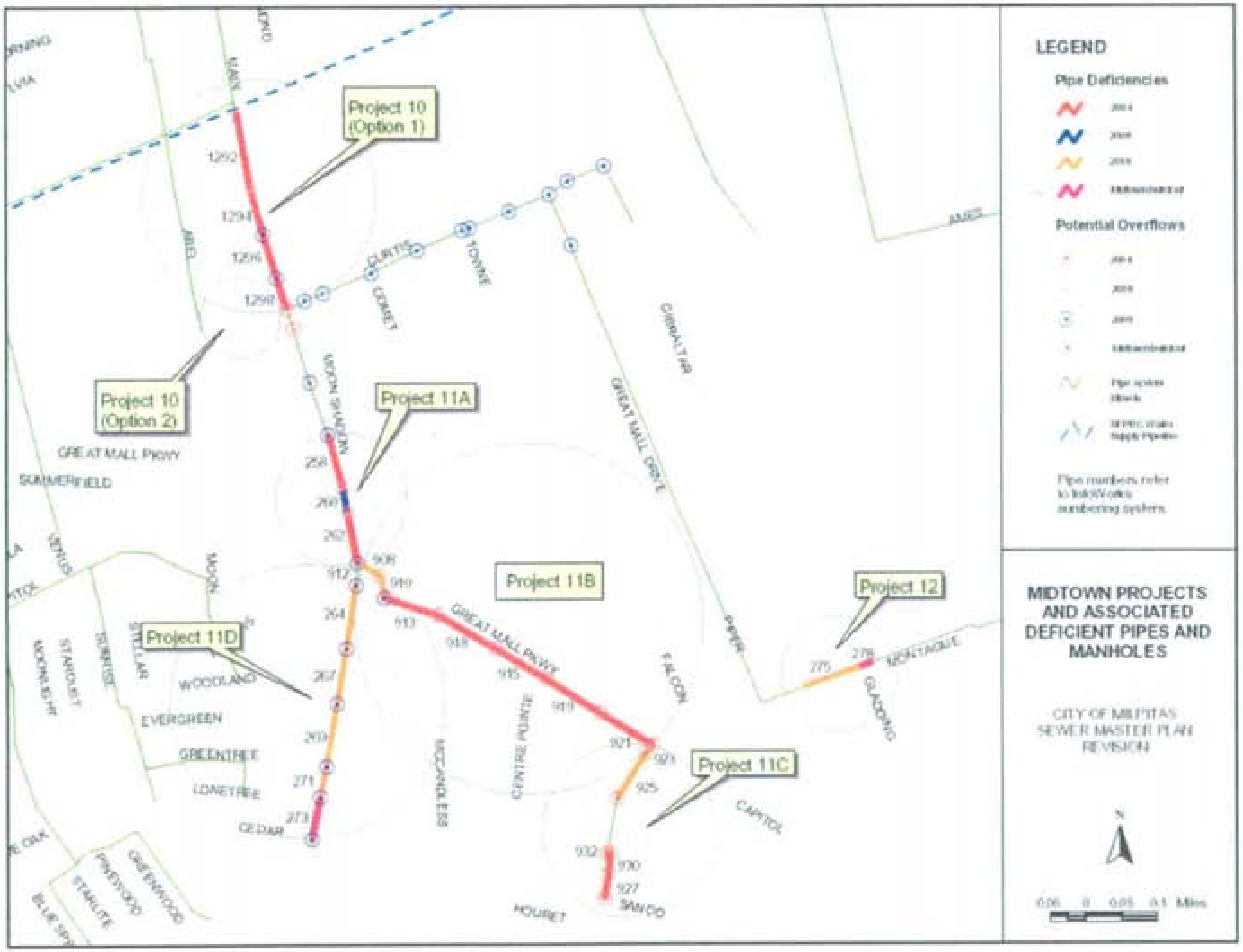
PIPE ID (G_ID/ SY_NAME) [a]	DIAMETER (in)		LENGTH (ft)	TYPE	COMMENTS
	Existing	Recommended			
275/49601	10	12	395	Replace	
278/61108	10	12	98	Replace	

a. Refers to HYDRA numbering system

6.4 Description of Pump Station Improvement Projects

As noted in Section 5.2.3, the Venus Way Pump Station has an existing firm capacity of 0.8 MGD. The estimated peak hour design flow through the pump station based on the HYRDA model was estimated at 0.3 to 0.5 MGD starting at 2008. The pump station has sufficient capacity to handle peak hour design flow at buildout; therefore, no improvements are necessary.

Figure 6-3: Proposed Improvements for Projects 10, 11 and 12



CHAPTER 7 **RECOMMENDATIONS**

Chapter Synopsis: This chapter consists of recommendations for implementation of the capital improvement projects developed in Chapter 6 to correct potential wet-weather conveyance deficiencies under existing, near- and long-term conditions. These recommendations include budget estimates and general scheduling targets for project implementation. This chapter also includes recommendations related to dry weather wastewater flow capacity needs at the WPCP, siphon maintenance, and HYDRA model update.

7.1 Capital Improvement Projects

Ten sewer capacity improvement projects have been identified as described in Chapter 6. The recommended sewer improvement projects are summarized in Table 7-1 and shown in Figure 7-1.

The following five timing categories are used in Table 7-1 and Figure 7-1:

- **Immediate** - Project should be undertaken as soon as possible. This priority is based on the potential for overflows associated with the project and/or the ratio of design flow to design capacity for the pipes in the project. Project 10 (Option 2) falls into this category.
- **Near-Term** - Project should be undertaken within the next two to four years. This priority is based on a capacity trigger for existing conditions. The capacity trigger is generally more than 150% of the design capacity of the pipe and the likelihood of system overflows is high based on the hydraulic gradeline calculations from the HYDRA model. Projects 11B and 11C fall into this category.
- **Mid-Term** - Project should be undertaken in the next five to six years. This priority is based on a capacity trigger for 2008. The capacity trigger is generally less than 150% of the design capacity of the pipe and the likelihood of system overflows is low based on the hydraulic gradeline calculations from the HYDRA model prior to 2008. After 2008 the capacity trigger is generally more than 150% of the design capacity of the pipe and the likelihood of system overflows is high based on the hydraulic gradeline calculations from the HYDRA model. Project 11A falls into this category.
- **Long-Term** - Project should be undertaken in conjunction with the pace of actual development of the Midtown Specific Plan area (2018 planning scenario). Project 11D falls into this category.
- **Indefinite** – Projects are not recommended to be undertaken unless further development or project necessitates them. Project is included in the list of capital improvement projects, but not considered in the cash flow analysis. Projects 1, 2, 5A, 6A, and 12 fall into this category.

Street improvement and sewer replacement schedules and water capital improvement projects were considered to develop recommendations for implementation of the sewer capital improvement projects. Relevant recommendations are shown in Table 7-1.

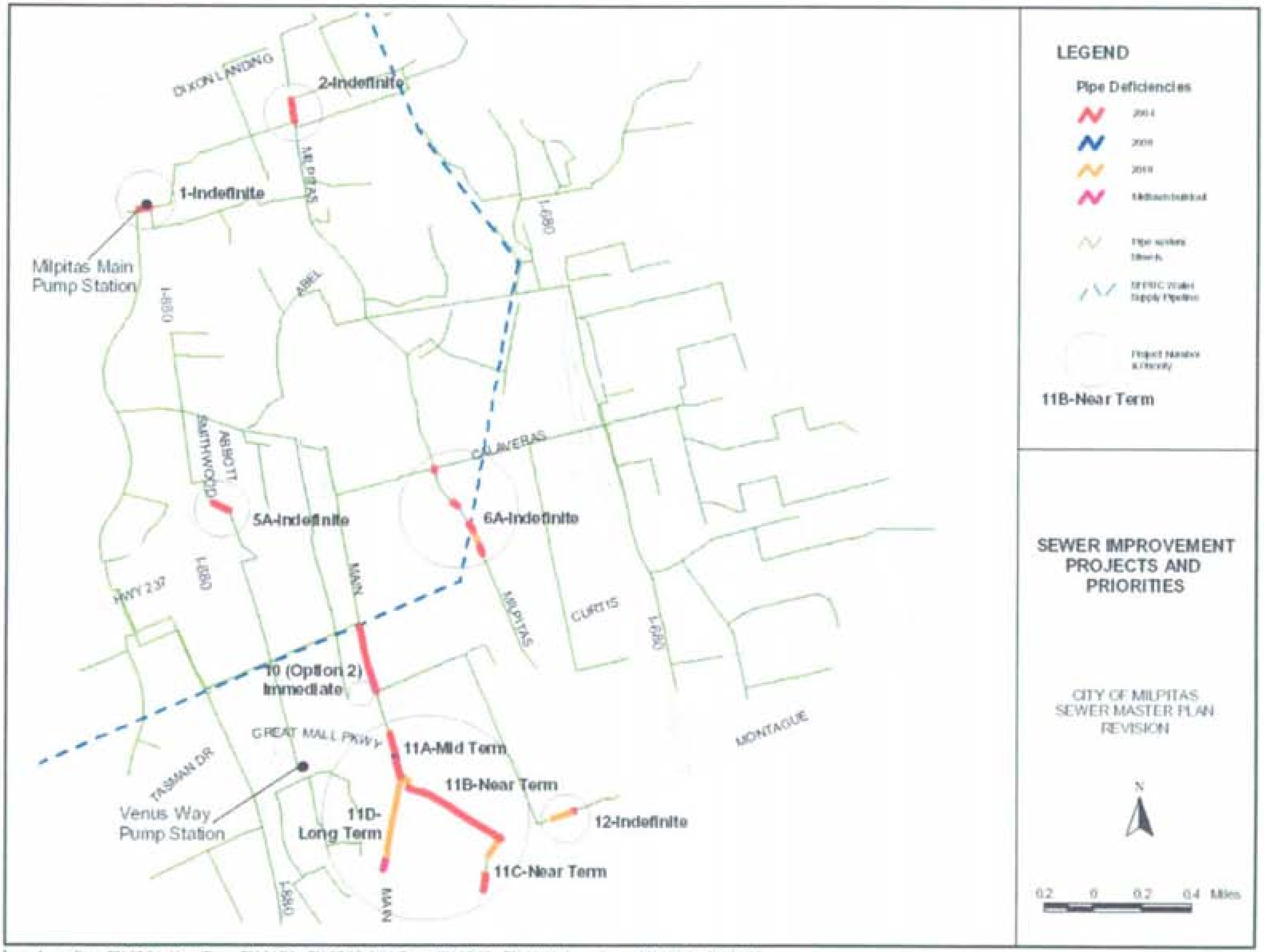
- **Street Improvement Schedule** - The City has an ongoing pavement replacement and improvement project. Since the recommended sewer improvement projects involve underground construction, it is recommended that the improvements be scheduled prior to pavement improvements. Project 10 (option 2) appear to coincide with street improvement projects.
- **Sewer Replacement Schedule** - The list of sewer projects proposed for replacement within less than 20 years per the Utility Depreciation Study (Schaaf & Wheeler, 2002) was reviewed to identify multipurpose projects. One reach of Project 1 (pipe under I-880) is identified in this study as exceeding its estimated 25-year life expectancy within the timeframe of this Master Plan Revision.

Table 7-1: Recommended Capital Improvement Projects

PROJECT	LOCATION	DESCRIPTION [a]	ESTIMATED CAPITAL COST ($1,000) [b]	COMMENTS/RECOMMENDATIONS
IMMEDIATE (FY 04/05)				
10 (Option 2)	South Main Street north of East Curtis Avenue (Curtis Avenue between South Main Street and South Abel Street)	• Construct diversion at N Main St and Curtis Ave • Construct 625 LF of 18-inch diameter sewer between S Main St and S Abel St	305	▪ A key design consideration at this location is access for the fire station at the intersection of South Main Street and West Curtis Avenue. ▪ This project is already a component of the City's downtown renovation plan.
NEAR-TERM (FY 05/06 – FY 07/08)				
11 B	Great Mall Project (Great Mall Parkway between South Main Street and Montague Expressway)	• Replace 360 LF of 15-inch with 18-inch diameter sewer • Replace 1,820 LF of 10-inch with 18-inch diameter sewer • Replace 450 LF of 10-inch with 15-inch diameter sewer	1,245	▪ Bottleneck in downstream pipes could result in upstream overflows. ▪ Street improvement project schedule for 2003-2004. Consider deferring paving project until after sewer project has been implemented.
11 C	Great Mall Project (Montague Expressway)	• Replace 885 LF of 10-inch with 12-inch diameter sewer • Replace 30 LF of 8-inch with 15-inch diameter sewer • Replace 325 LF of 8-inch with 12-inch diameter sewer	385	▪ Bottleneck in downstream pipe could result in upstream overflows.
MID-TERM (FY 08/09 – FY09/10)				
11 A	Great Mall Project (South Main Street north of Great Mall Parkway)	• Replace 590 LF of 18-inch with 27-inch diameter sewer • Replace 370 LF of 12-inch with 27-inch diameter sewer	705	▪ Bottleneck in downstream pipe could result in upstream overflows.
LONG-TERM (FY 10/11– FY 17/18)				
11 D	Great Mall Project (South Main Street south of Great Mall Parkway)	• Replace 1,515 LF of 8-inch with 12-inch diameter sewer • Replace 269 LF of 8-inch with 10-inch diameter sewer	640	▪ Capacity improvements should be reconfirmed according to actual Midtown Specific Area development.
INDEFINITE[c]				
1	I-880 Crossing	• Replace 225 LF of 18-inch with 27-inch diameter sewer	450	▪ The pipe has approximately 13 feet of cover and there are no anticipated overflows under all scenarios.
2	North Milpitas Boulevard near Jason Avenue and Homme Way	• Replace 490 LF of 8-inch with 10-inch diameter sewer	135	▪ The pipe has approximately 10 feet of cover and there are no anticipated overflows under all scenarios.
5A	Smithwood Street near Abbott Boulevard	• Replace 385 LF of 15-inch with 21-inch diameter sewer	220	▪ The pipe has approximately 7 feet of cover and there are no anticipated overflows under all scenarios.
6A	South Milpitas Boulevard between Calaveras Boulevard and Turquoise	• Replace 152 LF of 12-inch with 18-inch diameter sewer • Replace 595 LF of 12-inch with 15-inch diameter sewer	490	▪ The pipe has approximately 8 feet of cover and there are no anticipated overflows under all scenarios.
12	Montague Expressway west of Gladding Avenue	• Replace 495 LF of 10-inch with 12-inch diameter sewer	160	▪ The surcharge in these pipelines are minimum and are not observed until 2018 and buildout.
		GRAND TOTAL	**4,735**	

a. Additional details and local maps are provided in Chapter 6, Section 2. Length of pipe is expressed in Linear Feet (LF) and is rounded to the nearest 5 feet.
b. Expressed in FY 03/04 dollars. Rounded to the nearest $5,000. See details of cost estimation in Chapter 6, Section 2
c. Projects are not recommended to be undertaken unless further development or project necessitates them.

Figure 7-1: Recommended Capital Improvement Projects



1 Immediate: FY 03/04; Near-Term: FY 04/05 - FY 07/08; Mid-Term: FY 07/08 - FY 10/11; Long-Term: FY 10/11 – FY 17/18

- **Water Capital Improvements Projects** – The list of water capital improvement projects established in the 2002 Water Master Plan (RMC, 2003) was reviewed. No water project is located in the same area than the sewer capital improvement projects.

The projected budget (adjusted for annual inflation of 4%) for the ten recommended sewer conveyance capacity improvement projects is summarized in Table 7-2.

Table 7-2: Cash Flow Analysis

PROJECT	PRIORITY	PROJECTED BUDGET NEEDS ($1,000) [a]			
		FY 04/05	FY 05/06 – FY 07/08	FY 08/09 – FY 09/10	FY 10/11 – FY 17/18
1	Indefinite	0	0	0	0
2	Indefinite	0	0	0	0
5A	Indefinite	0	0	0	0
6A	Indefinite	0	0	0	0
10 (Option 2)	Immediate	320	0	0	0
11 A	Mid-term	0	0	890	
11 B	Near-term	0	1,460	0	
11 C	Near-term	0	450	0	
11 D	Long-term	0	0	0	1,110
12	Indefinite	0	0	0	0
TOTAL ($1,000)		320	1,910	890	1,110

a. Dollar amounts have been adjusted for annual inflation of 4%. Rounded to the nearest $10,000.

7.2 Additional Recommendations

The following are recommendations that were developed related to the other objectives of this Master Plan Revision.

7.2.1 Dry Weather Capacity Needs at the WPCP

As noted in Section 5.1, depending on a number of factors (including actual timing and type of redevelopment), the City may exceed the capacity allotment at the WPCP as early as 2015. Therefore, it is recommended that the City closely monitor its peak week dry weather flow. Depending on the peak week dry weather flow trend within the next ten years, the City should decide to engage negotiations with the WQCP and partner cities and districts on its allocated dry weather capacity.

7.2.2 Siphon Maintenance Issues

After comparing the costs of continued siphon cleaning to other solutions (e.g. lift stations, parallel pipelines, screw conveyors, grease trap/digesters), it is recommended that the City continue servicing the siphons according to the current maintenance schedule. The correctional facility's installation of the grinders is an example of what facilities causing discharge problems could do to help reduce the cleaning frequency of these siphons. Should the screw conveyor technology become more common in the U.S., it could be a potential solution to be explored further.

7.2.3 HYDRA Sanitary Sewer System Hydraulic Model

The HYDRA model used for this Master Plan provides the City with a valuable tool for analyzing the capacity of the sewer system at a planning level. The model can also be used to test the impact of development plans proposed in the City. The HYDRA model should be updated periodically to reflect changes in the sewer system (new sewer construction and any development) and revised flow information.